HARGIS

An Investigation of Web Stresses in Reinforced Concrete Beams

Theoretical and Applied Mechanics

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AN INVESTIGATION OF WEB STRESSES IN REINFORCED CONCRETE BEAMS

BY

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I HEREBY RECOMMEND THAT THE THESIS PREPARED UNDER MY SUPERVISION BY

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BE ACCEPTED AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE DEGREE OF Master of Science in Theoretical and Applied Mechanics

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WEB STRESSES

IN

REINFORCED CONCRETE BEAMS

TABLE OF CONTENTS

INDEX TO

TABLES, DIAGRAMS, DRAWINGS, AND PHOTOGRAPHS

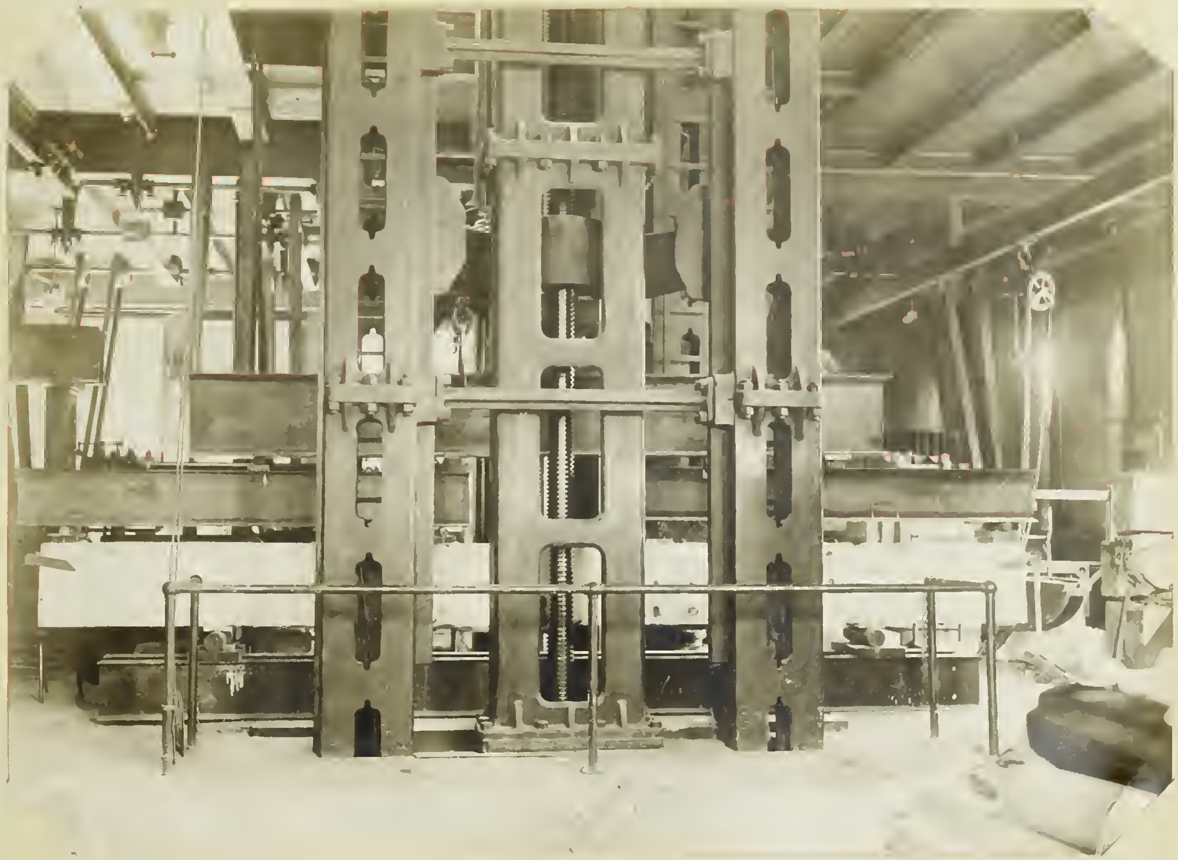
Tables

Diagrams

Drawings

Photographs

Photographs (Cont'd) Page



VIEW OF BEAM IN MACHINE

I INTRODUCTION

Present Theory Regarding Web Reinforcement.—Almost from the beginning of the use of reinforced concrete, the necessity for some means of reinforcing the web of members subject to flexure has been recognized, and numerous systems have been employed for this purpose. The low tensile strength of concrete has very clearly emphasized the importance of combined tensile and shearing stresses in concrete members subject to flexure, whereas in similar members of homogeneous material such as wood and steel, combined stresses are of little importance because of their higher tensile strength. The usual method of determining the amount of the maximum diagonal tension in a homogeneous beam at any point gives

$$t = \frac{1}{2} s + \sqrt{\frac{1}{4} s^2 + v^2}$$

in which t = maximum diagonal tensile unit stress, s = longitudinal tensile unit stress at that point, and v = horizontal or vertical shearing unit stress at the same point. The direction of this maximum diagonal tensile stress makes an angle with the horizontal equal to one-half the angle whose cotangent is $\frac{1}{2}\frac{s}{v}$. When $s = 0$ as is the case at the neutral surface, $\frac{1}{2}\frac{s}{v} = 0$ and the maximum diagonal tensile stress makes an angle of 45° with the horizontal and has a value equal to v at the neutral surface. When $v = 0$, as is the case at the lower side of the beam, $t = s$ and makes an angle of 0° with the horizontal. In a reinforced concrete beam, the non-homogeneity of the concrete, and the presence of longitudinal steel modify the

above theory of combined stresses. The amount and nature of this modification are unknown, but the way in which cracks open in a concrete beam under test seems to indicate that the maximum diagonal tensile stresses act in approximately the direction indicated by the above formula. When the web is reinforced the cracks open in a different way and nothing is known of the exact direction and amount of the diagonal stresses.

The systems of web reinforcement employing steel inclined at approximately 45° and rigidly attached to the longitudinal tensile steel have proven most effective, but the difficulty and expense of rigid attachment in the field has limited the use of this type. The most common method of reinforcing the concrete web is by means of vertical stirrups anchored to the longitudinal steel by looping the closed end under the longitudinal rods at points of positive moment, and over the rods at points of negative moment. This is usually supplemented by bending some of the horizontal rods up at an angle of about 45°. Beams so reinforced have not developed as high shearing values as those with rigidly attached inclined web steel.

Very little stress is supposed to be carried by the web reinforcement until after diagonal cracks have formed. After these cracks have formed, the action then may be considered to be a combination of something in the nature of beam action and truss action. The proportion of the shear which is taken by one of these actions will depend upon the amount the crack has opened. The exact nature of this action is best demonstrated by Taylor and Thompson, their explanation being about as follows: Referring to Fig. 1 page 4 let the line <u>AB</u> represent the horizontal steel, <u>ac</u> and <u>bd</u> vertical stirrups spaced

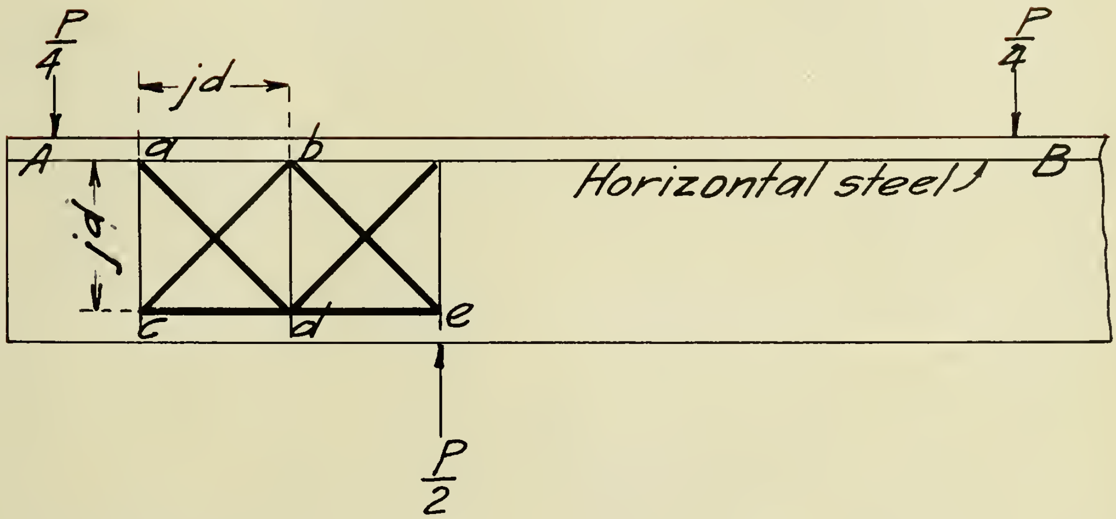
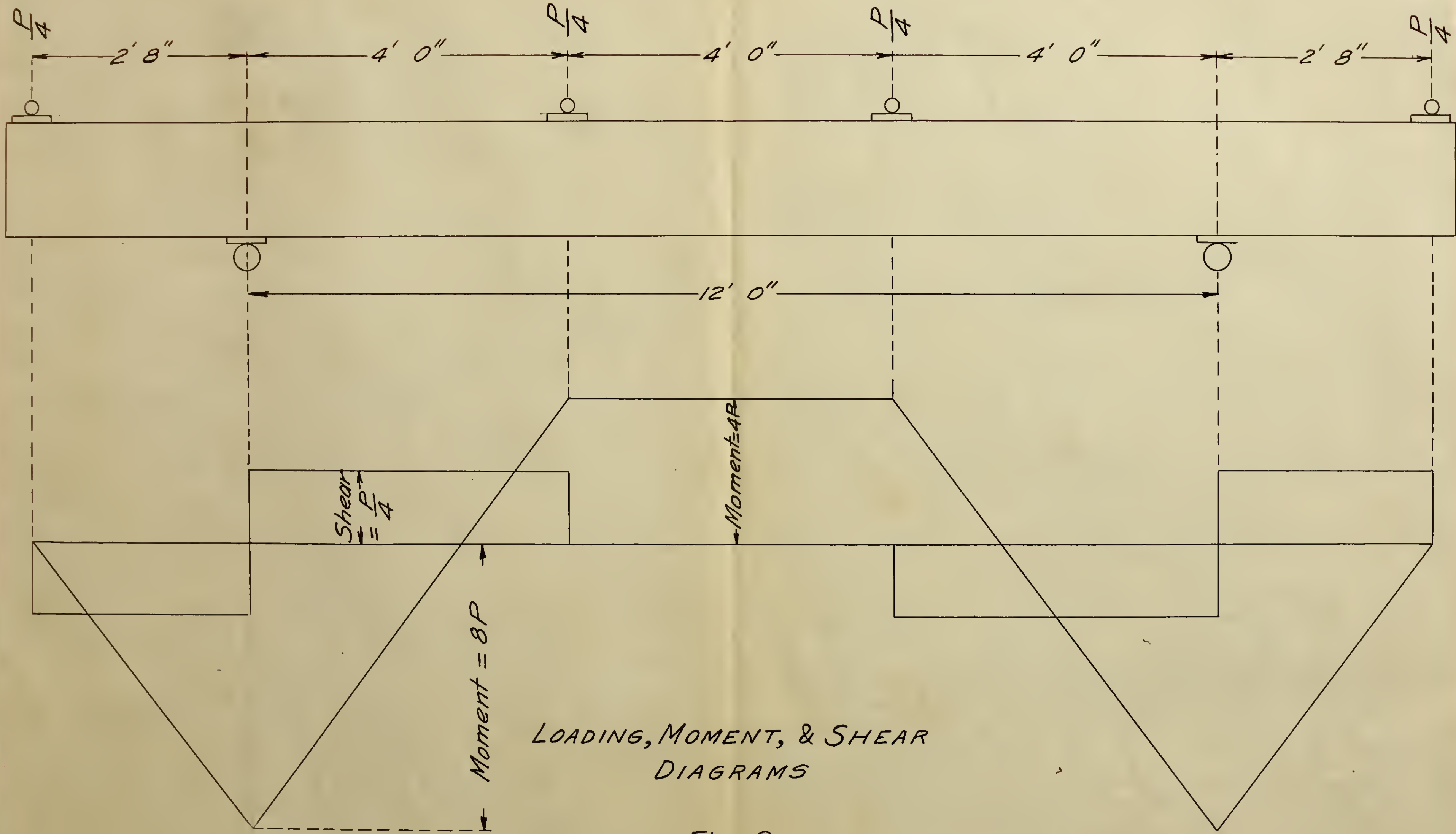


FIG. 1.

jd distance apart, and ce the centerline of the centroid of the compressive stresses. This may be assumed to be analogous to the action of a Howe truss, in which the horizontal steel is the lower chord (upper here), the concrete in the bottom of the beam the upper chord (lower here), and the concrete web between the stirrups the compression diagonals. To show that the stress in the stirrups is measured by the shear, consider the joint a. The tension in the stirrup at a is equal in magnitude to the horizontal component of the stress in the diagonal. This horizontal component is equal to the difference in the stress in the horizontal chord just to the left of a and that just to the left of b. But this change in stress is proportional to the change in bending moment. Considering any two points on this horizontal chord which are an infinitesimal distance apart, then $\frac{dM}{dx} = V$ or $dM = Vdx$ = change in bending moment over the infinitesimal distance. Now if the points are a definite distance apart, as a and b, the difference between the bending moment at a and the bending moment at b is equal to the external shear at this place times the length ab. Therefore, the difference between the total stress in the chord at the point a and the stress at the point b, as in any simple truss, is equal to the difference between the moments at these two points, which, as stated above, is Vjd divided by the depth of the truss jd, or in other words, the shear at this point = V. Hence this equals the total stress in the stirrup ac when the stirrup spacing = jd. For any other stirrup spacing s the stress in the stirrup would equal $V.\frac{s}{jd}$. If the spacing = $\frac{jd}{2}$ the stress carried by stirrup = $\frac{1}{2}$ as much, if s = $\frac{jd}{3}$ the stress in each stirrup = $\frac{1}{3}$ as much, and so on.



LOADING, MOMENT, & SHEAR DIAGRAMS

Fig. 2.

For inclined stirrups at an angle of 45°, the action is considered similar to a Pratt truss, the stress in the inclined steel being $0.7V \frac{s}{jd}$.

2 Scope of Investigation.—A total of eleven beams was tested, all of the same dimensions but having various arrangements of web reinforcement, as well as varying percentages of longitudinal reinforcement. All beams over-hung the supports 2 ft. 8 in. at each end, the supports being 12 ft. apart. The loading was applied at four points as shown by Fig. 2 so as to make the moment at the support twice the moment in the center of the span. The loading was such as to secure perfect restraint at the supports as calculated by the ordinary beam formula for the elastic curve. It is recognized that this is an assumption but it is felt that such an assumption is justifiable in the absence of more knowledge concerning the elastic properties of a reinforced concrete beam. Of these eleven beams, the middle portions of two of them were tested as simple beams after failure over the supports.

As stated before, it was the purpose to investigate the web stresses developed by making direct measurement of the stresses in the steel, and this method of investigation may be considered as pioneer in this field of investigation.

3. Acknowledgment.—These tests were made in the Laboratory of Applied Mechanics, University of Illinois, as part of the regular investigational work carried on by the Engineering Experiment Station during the spring of 1911.

The work was done under the general direction and supervision of Prof. A. N. Talbot who gave many helpful suggestions in

planning and conducting the tests; to Mr. W. A. Slater, First Assistant in the Engineering Experiment Station, is due much credit for his assistance in conducting the tests on such unweildy test specimens as these were; Mr. A. R. Lord, Research Fellow, rendered valuable assistance on the tests and in working up the data; Mr. D. A. Abrams, Associate in the Engineering Experiment Station, supervised the building of four of the beams and gave other assistance in connection with the tests. To these and other members of the staff due acknowledgment is made for assistance and suggestions connected with the tests.

The Corrugated Bar Co. of St. Louis furnished the corrugated unit frames used in four of the beams, the American System for Reinforcing, Chicago, furnished the unit frames used in beams 375.1 and 375.3 the latter of which is yet to be tested.

II

MATERIALS, TEST PIECES, AND METHODS OF TESTING

4. Materials and Their Properties.—The materials used in making the test specimens were all of the grade employed in first class building construction. The properties of the various materials are given in the following paragraphs.

Cement.—Two standard brands of portland cement were used. Universal was used for beams 371.2, 372.1, 373.1, 374.1, 375.1, 376.1, 376.2, 376.5, 376.6, and Lehigh for beams 372.2 and 373.2. Samples of the Universal cement were tested at various times during the progress of making the specimens. Only one test was made of the Lehigh cement. The following tables give the results of the tests, the values for the briquette strengths being the average of five briquettes tested.

Tensile Strength of Standard Briquettes in Pounds per sq. in.

Universal

	Sample No. 1		Sample No. 2		Sample No. 3		Sample No. 4	
Age Days	7	28	7	28	7	28	7	28
Neat	589	674	684	709	653	731	662	696
1:3 Standard Sand	198	278	227	283	240	319	214	282
1:3 Sand used in Beams	265	323						

Lehigh

Age Days	7	28
Neat	719	805
1:3 Standard Sand	248	329

Fineness Test

Universal

Sieve	Per cent Passing
50	98.9
100	96.5
200	82.5

The initial set of the Universal, as determined by the Vicat needle, was found to occur in 1 hour and 20 minutes, and hard set in 4 hours and 40 minutes.

No test to determine the fineness and time of set was made on the Lehigh cement.

Sand.—The sand used was torpedo sand from Attica, Indiana. It was of good quality, clean, and well graded. It combined with the cement used in a very satisfactory manner giving a higher briquette strength than did the same cement with standard Ottawa sand. It was from the same locality and of the same quality as the sand used in making reinforced concrete test specimens for the past several years at the University of Illinois.

Stone.—A good quality of rather hard limestone from Kankakee, Illinois, was used, the specifications accompanying the order requiring it to pass a 1-inch and be retained on a $\frac{1}{4}$-inch mesh. It is representative of the stone most used in reinforced concrete building construction in Illinois, and is the same as has been used in previous experimental work of the Station. No special tests were made to determine its voids.

TABLE I

TENSION TESTS OF STEEL FROM BEAMS

Steel From Beam No.	Average Diameter of Rod	Total Load at Yield Point	Average Unit Stress at Yield Point
372.1	.752	14 960	33 200
	.750	14 200	
372.2	.249	1 780	38 600
	.253	2 000	
	.253	1 740	
	.252	2 200	
373.1	.745	15 200	34 400
	.746	15 100	
	.751	15 100	
373.2	.749	14 870	34 200
	.751	15 200	
	.753	15 200	
373.2	.253	2 110	37 800
	.253	1 760	
	.252	2 070	
	.253	1 850	
	.254	1 720	
374.1	.759	15 300	35 100
	.753	16 300	
375.1	.583	15 900	60 500
	.624	19 000	
376.2	.715	25 510	56 600
	.715	25 350	
	.715	24 000	
	.202	1 390	41 350
	.200	1 250	
	.200	1 220	
	.200	1 360	
376.6	.715	24 200	54 800
	.713	24 100	
	.712	24 100	
	.25	3 300	60 200
	.25	3 570	
	.25	4 220	
	.25	4 275	

Proportions.—In all test specimens 1:2:4 concrete was used. The concrete was thoroughly mixed by hand on the concrete floor of the laboratory by men employed for this purpose for the past few years. It was also done under the supervision of some member of the Station staff in order to insure a uniform mix. A fairly wet mix was used.

Steel.—Specimens were cut from the longitudinal steel of all the beams tested. Pieces taken from the lot of $\frac{1}{4}$-inch round steel used in making the stirrups for beams 371.2, 372.1, 372.2, 373.1, and 373.2 were tested and the yield point is shown in table I page 11 opposite 372.2 and 373.2. It will be noted that the yield point is higher than for the $\frac{3}{4}$-inch round bars used in the same beams. This is believed to be due to the fact that the 100 000 lb. testing machine used did not indicate the yield point as accurately for the $\frac{1}{4}$-inch rods as it did for the $\frac{3}{4}$-inch rods. It was the impression at the time that the $\frac{1}{4}$-inch steel was of the same quality as the $\frac{3}{4}$-inch rounds. The longitudinal steel used in beams 376.1, 376.2, 376.5, and 376.6 were round corrugated bars furnished by the Corrugated Bar Co. The web reinforcement in 376.1 and 376.2 was smooth round rods 0.21 inch in diameter, and that used in 376.5 and 376.6 was $\frac{1}{4}$-inch square corrugated bars. In the table will be found the yield points of the web steel used in the corrugated unit frames. It will be noted that the yield point of the 0.21 inch round steel was much lower than that of the $\frac{3}{4}$-inch rods in the same beam. The use of the 100 000 lb. machine may have affected the results slightly. The 0.21 inch rounds used in 376.1 and 376.2 was not drawn but rolled steel. All steel used in 375.1 was plain round bars of high yield point as shown by the table.

5. <u>Test Beams</u>.—All the beams were 18 feet long by 15 x 8 in. effective cross-section, the overall depth being 17 in. The total length of the beams was approximately 8 in. more than the distance between the extreme end load points. The details of the reinforcement are shown in the drawings for the various beams, as well as in table V page 117. The beams were made relatively deep in order to emphasize the web stresses developed. The bent up longitudinal rods were so arranged that the inclined portion would pass approximately through the point of contraflexure. The unit frames used and beam 374.1 contained rods running the entire length near the bottom of the beam. The others also had the rods so arranged that the compressive portion of the cross-section at some points contained steel. In the case of these latter beams, it was necessary to carry the steel into the compressive region in order to provide the required length for anchorage against slipping. In the case of the unit frames the rods running straight for the entire beam length were needed to fasten the web reinforcement to. It will be noted that in many cases the stirrups did not have a snug fit against the longitudinal bars, and it may be said that this was true in most cases since such a connection was not practicable and would seldom be attained in practice. It will be further noticed that the stirrups of beams numbered 371.2, 372.1, and 372.2 were anchored to the horizontal steel in a different way than were those of beams 373.1 and 373.2. It was the intention to have all anchored like those of beams 373.1 and 373.2 but by a misunderstanding this was not done. It was feared that this method of anchorage would prove a weakness, but, as discussed later, such weakness did not occur under the stresses developed in the stirrups. It will be noted that

VIEW OF CORRUGATED BAR CO'S UNIT FRAME WITH ONE OF THE BEAMS AS A BACK-GROUND

the stirrups of beam 373.1 had the ends bent through 180° for the purpose of anchorage against slipping, while 373.2 had no such provision but the unbent ends extended to the horizontal faces of the beam. This was done in order to enable the detection of any slip of these stirrup legs by placing Ames dials against them. Another feature of 373.1 was the space of about one-half inch purposely left between the closed end of the stirrups and the horizontal steel. This was done in order to detect any possible weakness of such construction.

Beam 374.1 had no web reinforcement and was included in order to have some basis of comparison between the web resistance of this size beam without web reinforcement on the one hand, and with web reinforcement on the other.

It will be noticed that practically the only difference between the series 371, 372, and 373 is in the ratio of the per cent of steel at the center of the beam, and the per cent over the supports. This difference was made in order to study the effect of this variation upon the web stresses.

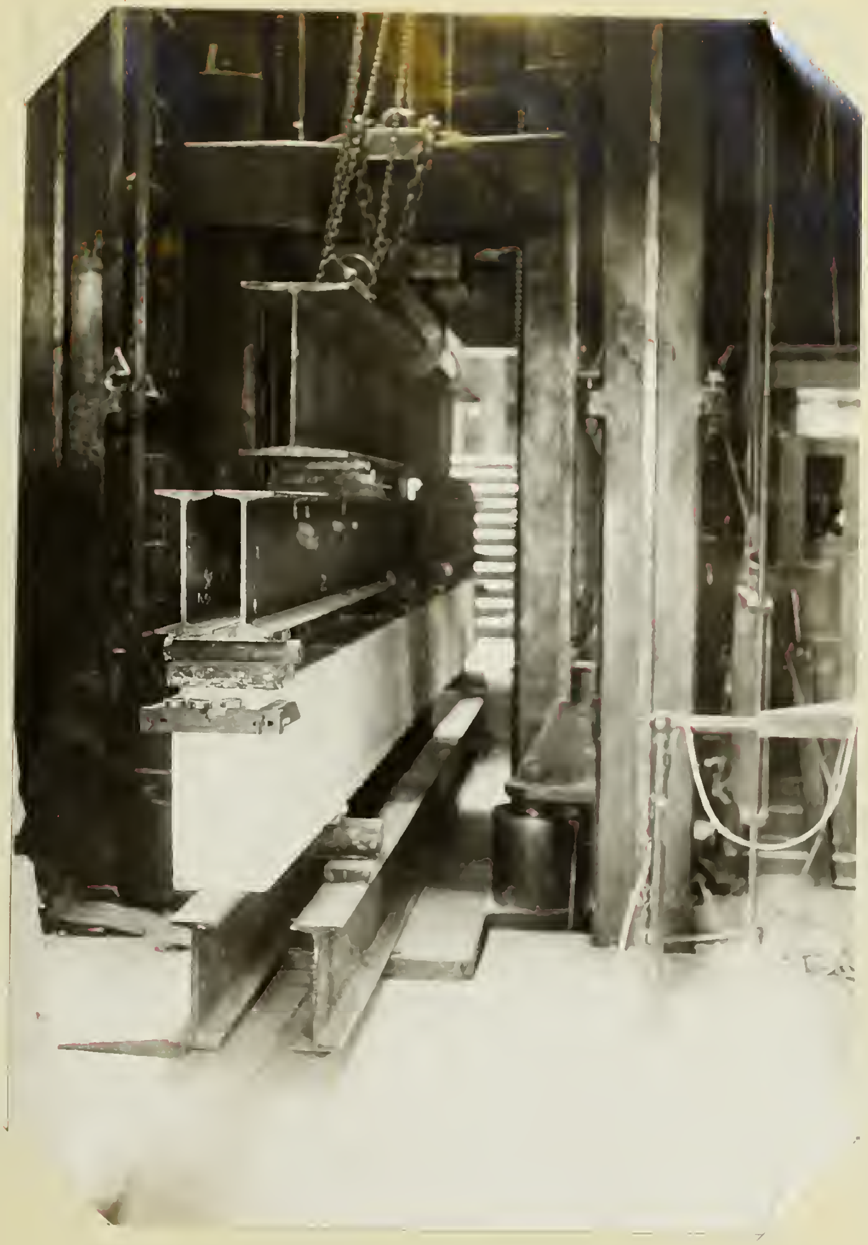
6. Making of the Beams.—The beams were made in the same way as described in the University of Illinois bulletins on reinforced concrete beams. A strip of building paper was placed on the concrete floor and the wooden forms were placed on this paper. The presence of so many rods permitted tamping only with small tamps such as a 5/8-in. rod or a 1/2 x 2-in wooden strip. The tamping in the case of beams 371.2, 372.1, 372.2, 373.1, and 373.2 could not be quite as thorough as in the case of the others because of the difficulty of keeping the loose stirrups in place during

thorough tamping. The steel was supported until the concrete was stiff.

7. Auxiliary Test Specimens.--In order to know the general character of the concrete entering into the beams, three 6-in. cubes and one 6 x 8 x 40-in. control beam were made from concrete taken from the center of the batch entering into the construction of each beam. The cubes were for the purpose of determining the compressive strength, and the control beams for obtaining the modulus of rupture which is an index to the tensile strength of the concrete.

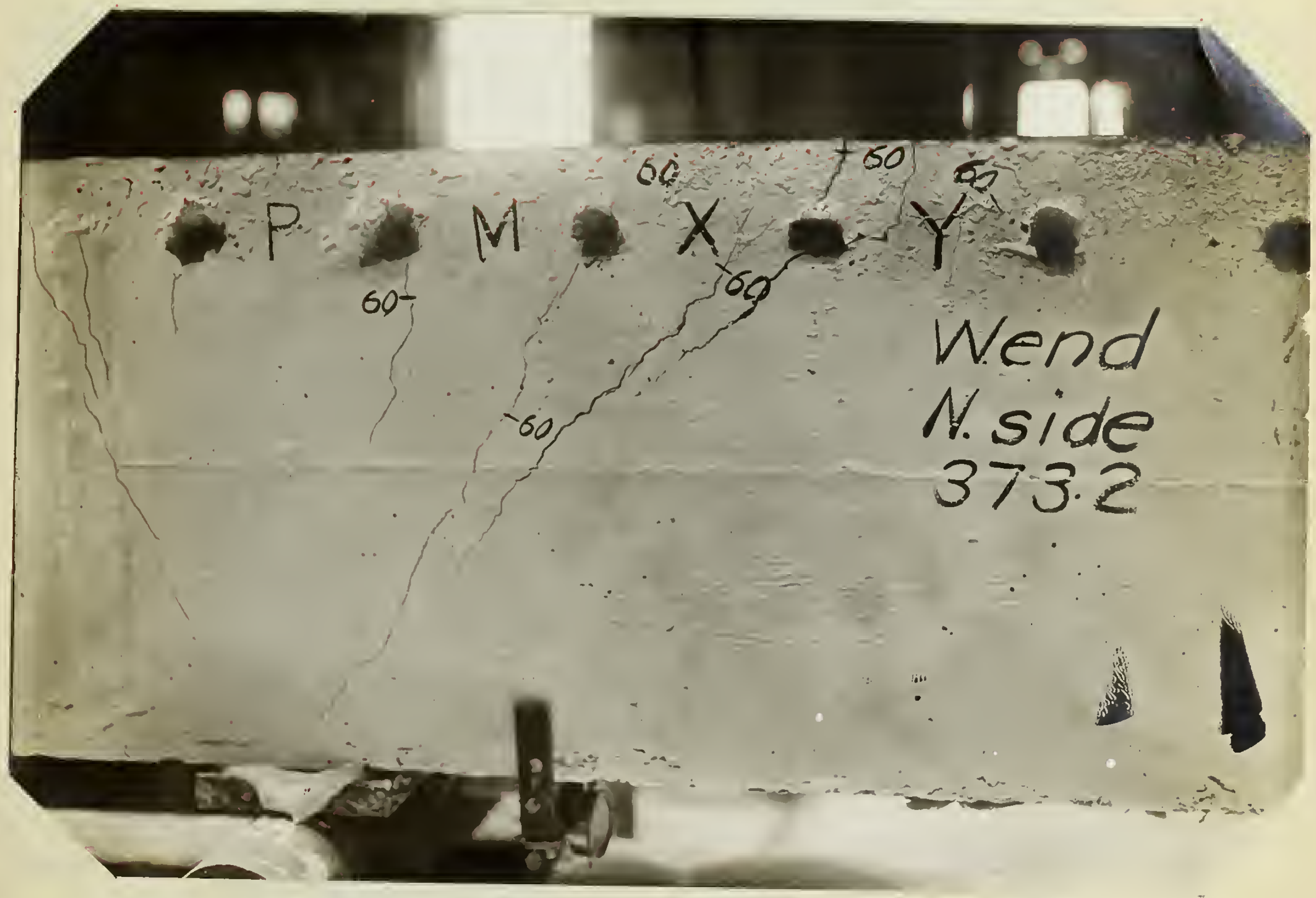
8. Storage.--The forms were left on the beams for one week, after which the beams were sprinkled daily with a hose. However, at times, due to the congestion of test specimens in the concrete laboratory, and other causes, some beams were sprinkled more than others. The cubes were removed from their molds after about 7 days and then buried in moist sand. The control beams were treated in the same manner as the large beams. From the conditions of storage it can be seen the strength of the auxiliary specimens cannot be taken as an absolute index to the quality of the concrete in the large beams. All specimens were tested at approximately the age of 60 days, the exact age being given in table V page 117. The auxiliary specimens were tested at approximately the same dates as the large beams. The temperature of the atmosphere varied from 60° to 70° Fahrenheit, as shown by the daily records kept. This does not include any night temperatures as no records of this were kept.

9. Methods of Testing.--The 600 000 lb. testing machine was used in testing the beams. The photograph page 1 shows the general arrangement. The end view of one of the beams in the ma-



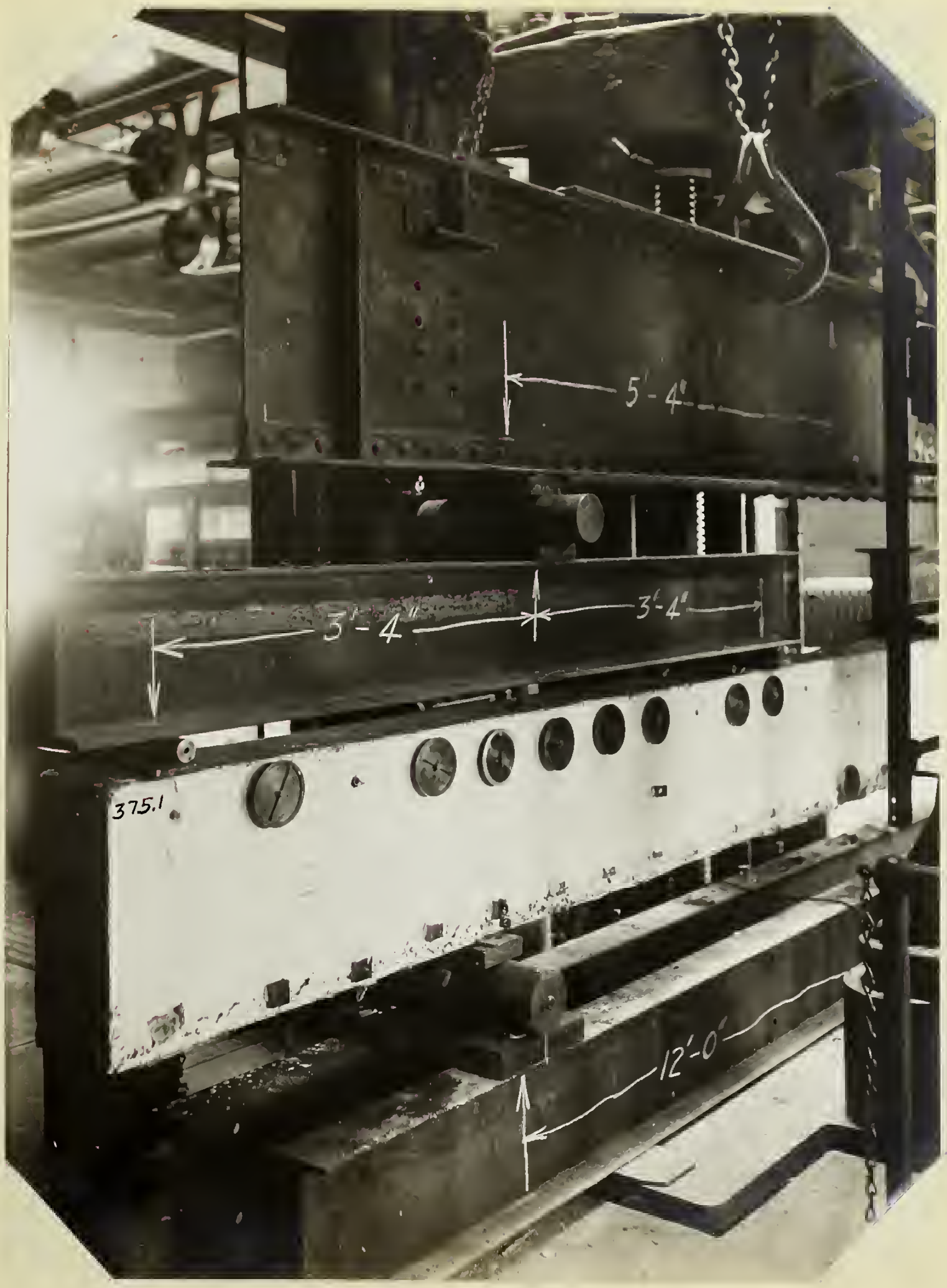
END VIEW

SHOWING METHOD OF LOADING THE BEAMS



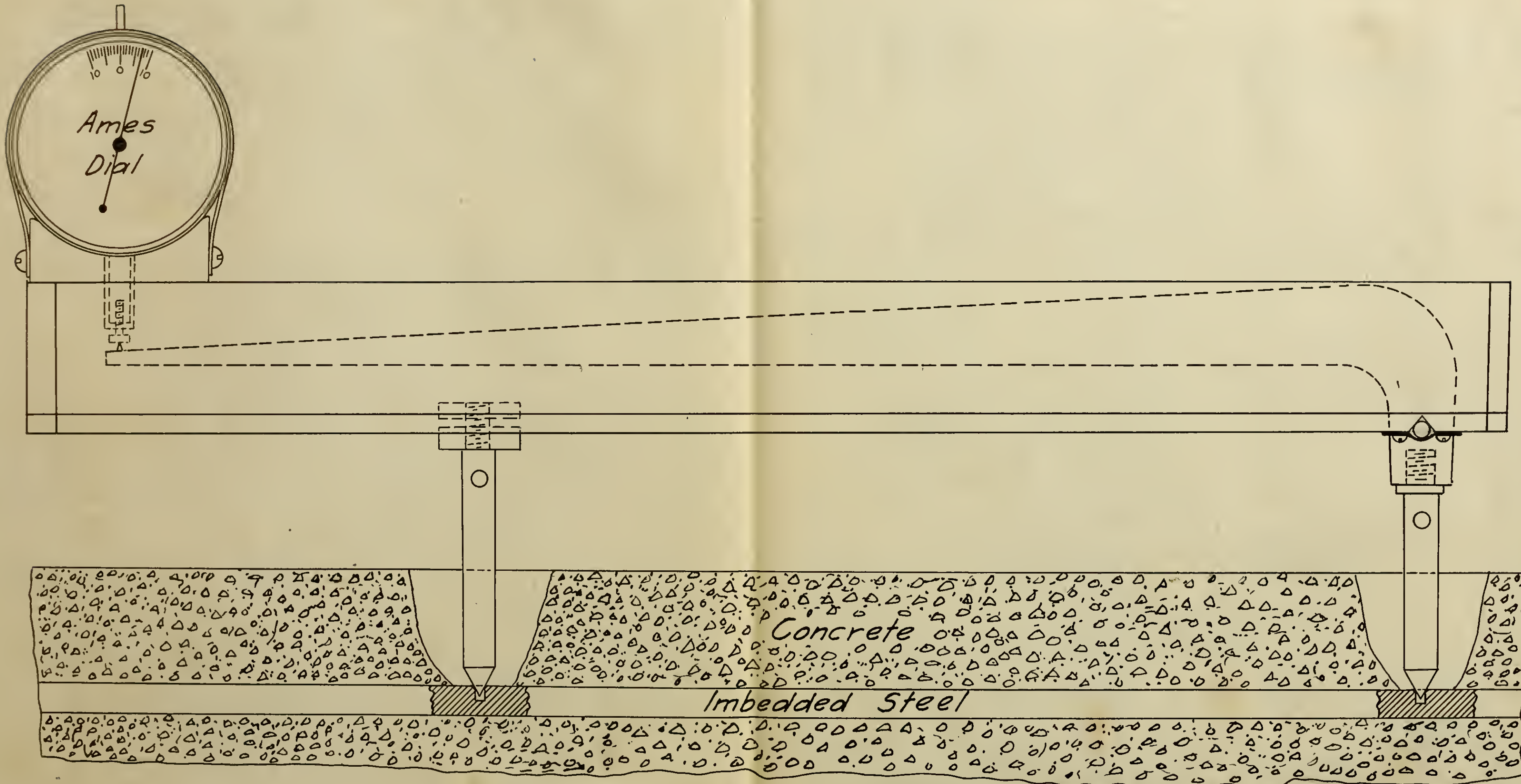
METHOD OF APPLYING AMES DIAL TO LEG OF STIRRUP

Only the portions of the cracks which crossed the gage lengths were marked.



VIEW SHOWING THE USE OF WIRE WOUND DIALS ON

BEAMS 375.1 AND 374.1



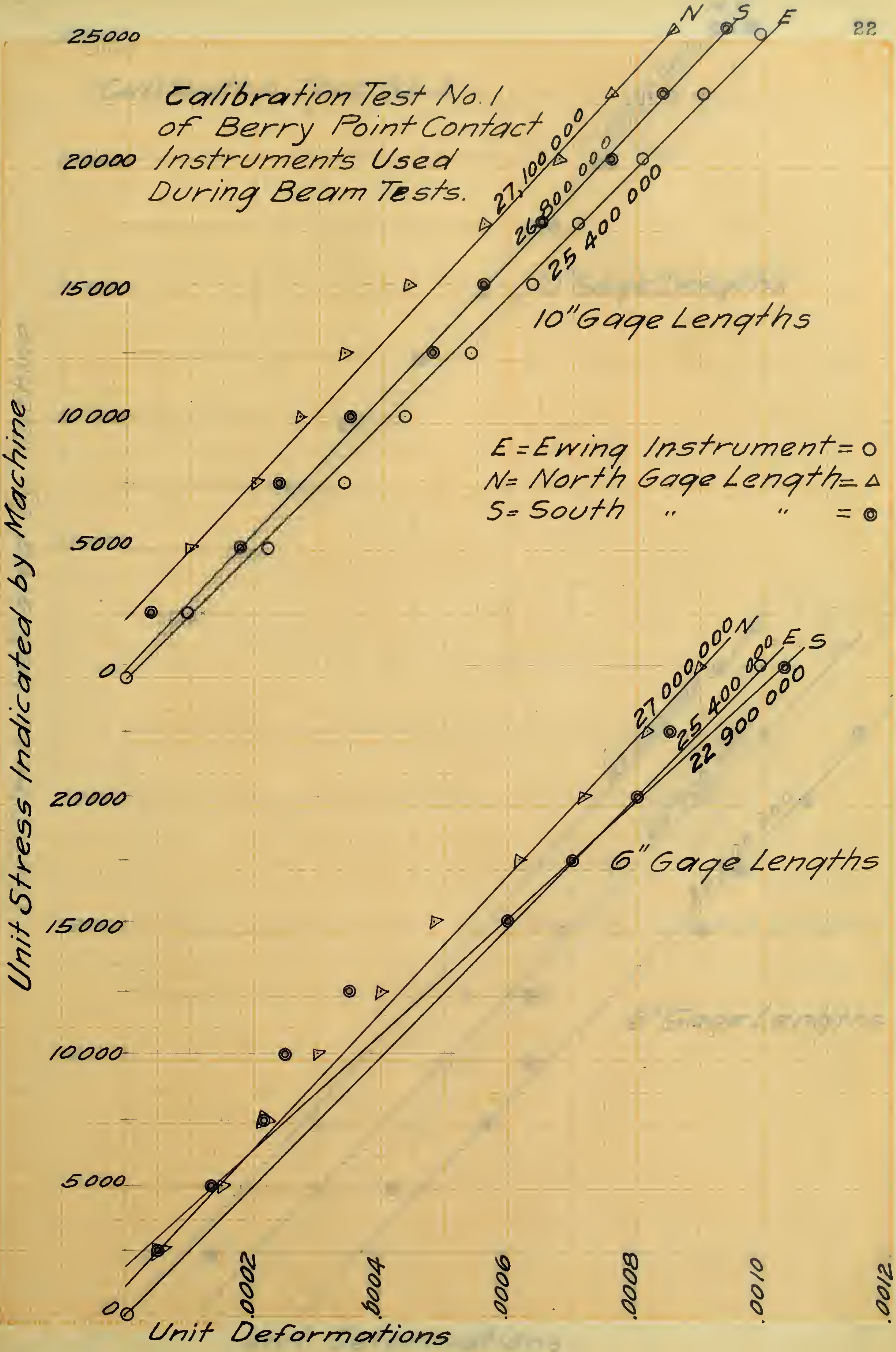
BERRY EXTENSOMETER

Full Size

Fig. 3.

chine shows the method of applying the load. The points of support and application of load were as shown by Fig. page . At these points steel plates were used to transmit the load to the beam in order to prevent crushing. Plates 1 inch thick and 6 inches wide were used on all beams except numbers 374.1, 375.1, 376.1, and 376.2 on which 4 inch plates were used at the points of support and 6 inch plates were used at the load points. The use of 4 inch plates may have caused the crushing noted elsewhere in connection with 376.1 and 376.5. The plates were set in a bed of plaster of Paris. The load was then applied to these plates by means of steel rollers. Care was taken to eliminate an eccentric application of the load, but it is believed that there was some eccentricity in each case, although there was no means of knowing how much. Furthermore, several of the beams were somewhat crooked due to the use of warped forms. The slowest speed of the machine, 0.05 inch per minute, was used in applying the load. The loads were, as a rule, applied in increments of 15 000 lb. until a load of 60 000 lb. was reached, after which increments of 20 000 lb. were used. The load was held while the instrument readings were taken, after which the next increment was applied without any release of load. The weight of the beam itself was neglected in the calculations, but the weight of the loading steel I-beams,= 2 300 lb.,was used and called the initial load.

The new modified Berry extensometers were used in measuring the deformation of the steel of all beams excepting 374.1 and 375.1. These instruments had not been made when these were tested, hence the wire wound dials were used as shown on page 19. It is not believed that the measurements taken with the wire wound dials mean much, since it assumes that the concrete deforms with the steel


Calibration Test No. 1
of Berry Point Contact
Instruments Used
During Beam Tests.
10" Gage Lengths
27,100 000
26,800 000
25 400 000
N
S
E
E = Ewing Instrument = ○
N = North Gage Length = △
S = South " " = ◎
6" Gage Lengths
27 000 000 N
25 400 000 E
22 900 000 S
Unit Stress Indicated by Machine
25000
20000
15000
10000
5000
0
20000
15000
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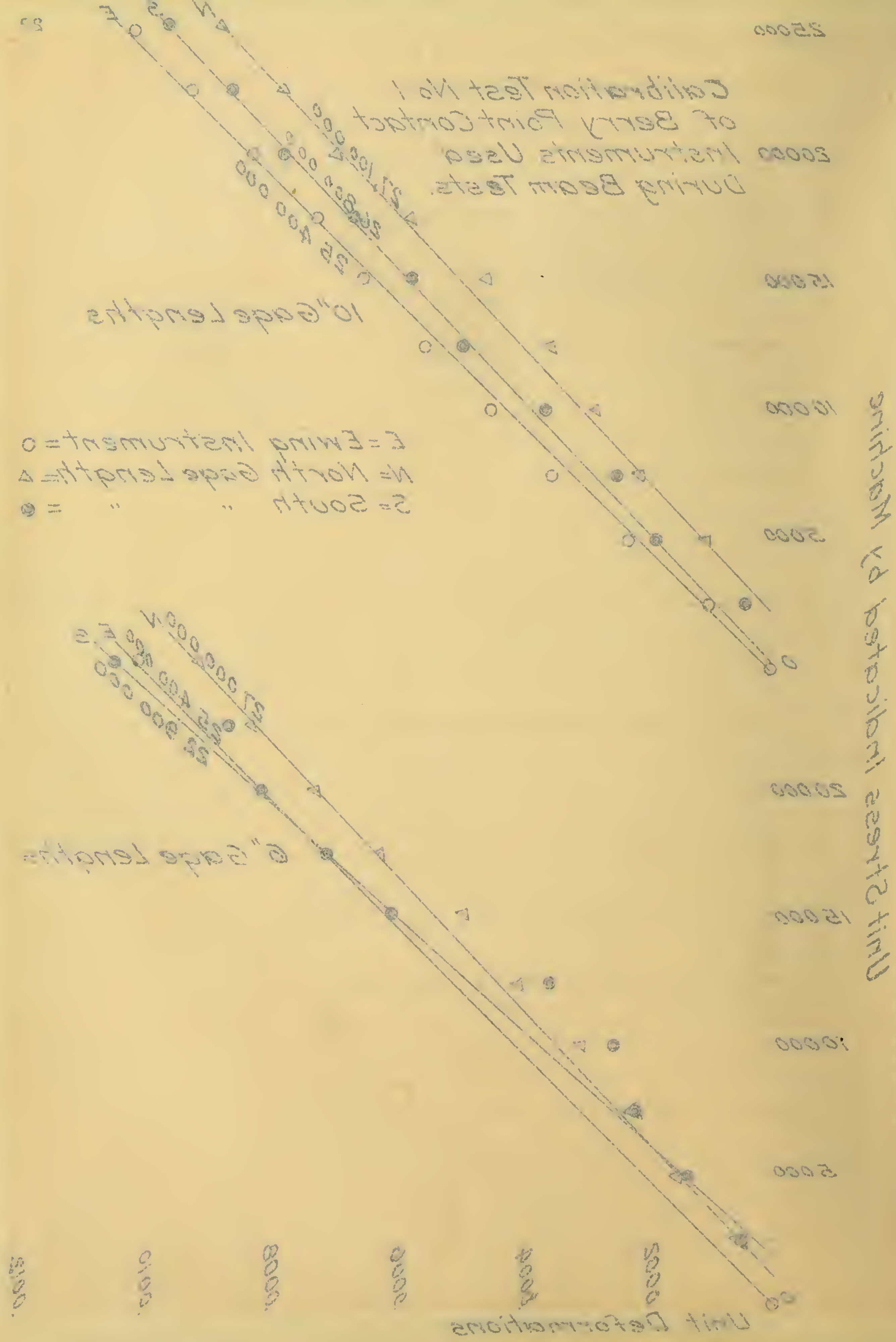
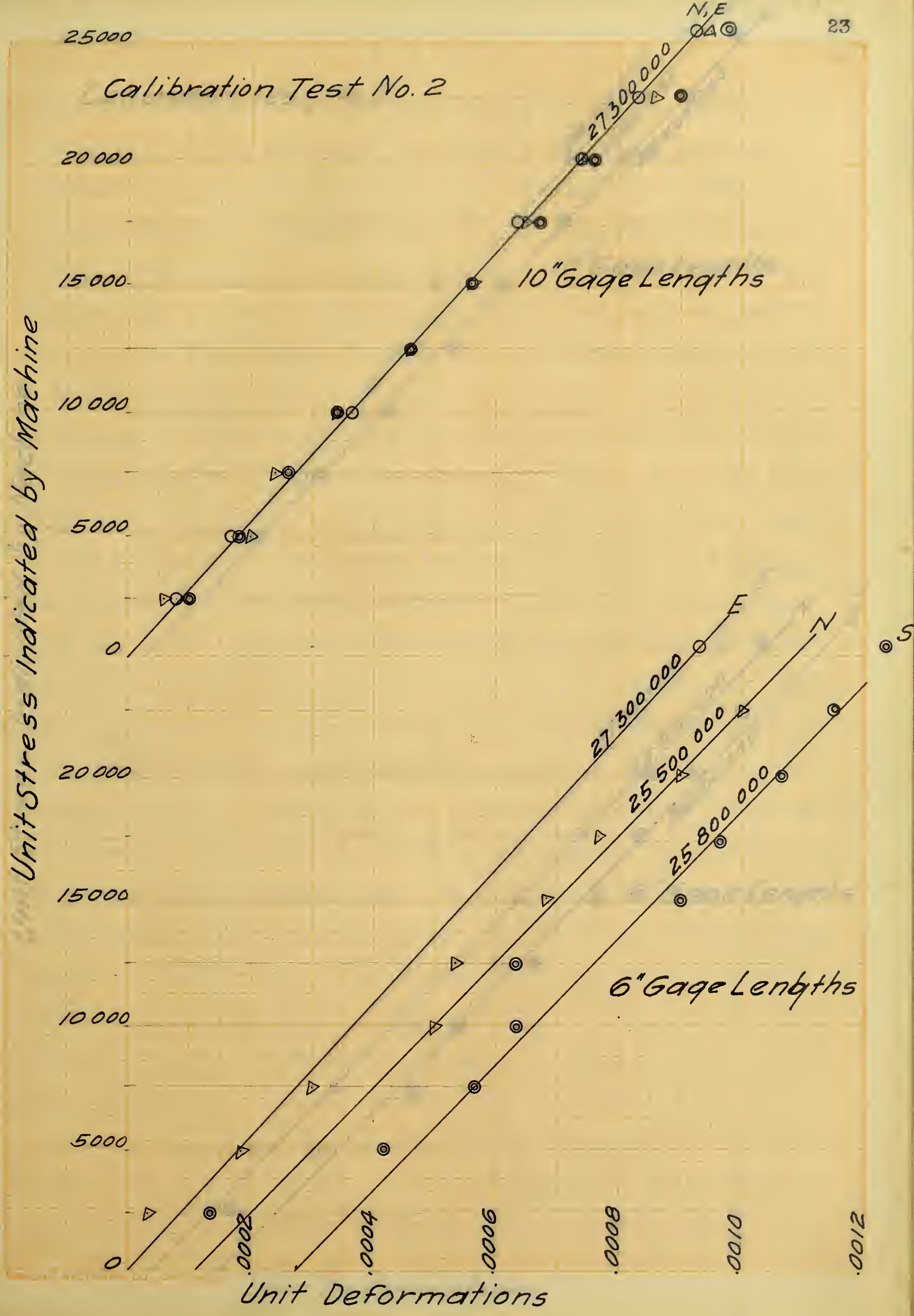
5000
0
0
.0002
.0004
.0006
.0008
.0010
.0012
Unit Deformations



Calibration Test No 1
of Berry Point Contact
Instruments Used
During Beam Tests.
10" Gage Lengths
6" Gage Lengths
E = Ewing Instrument = o
N = North Gage Length = △
S = South " " = ●
Unit Stress indicated by Machine
Unit Deformations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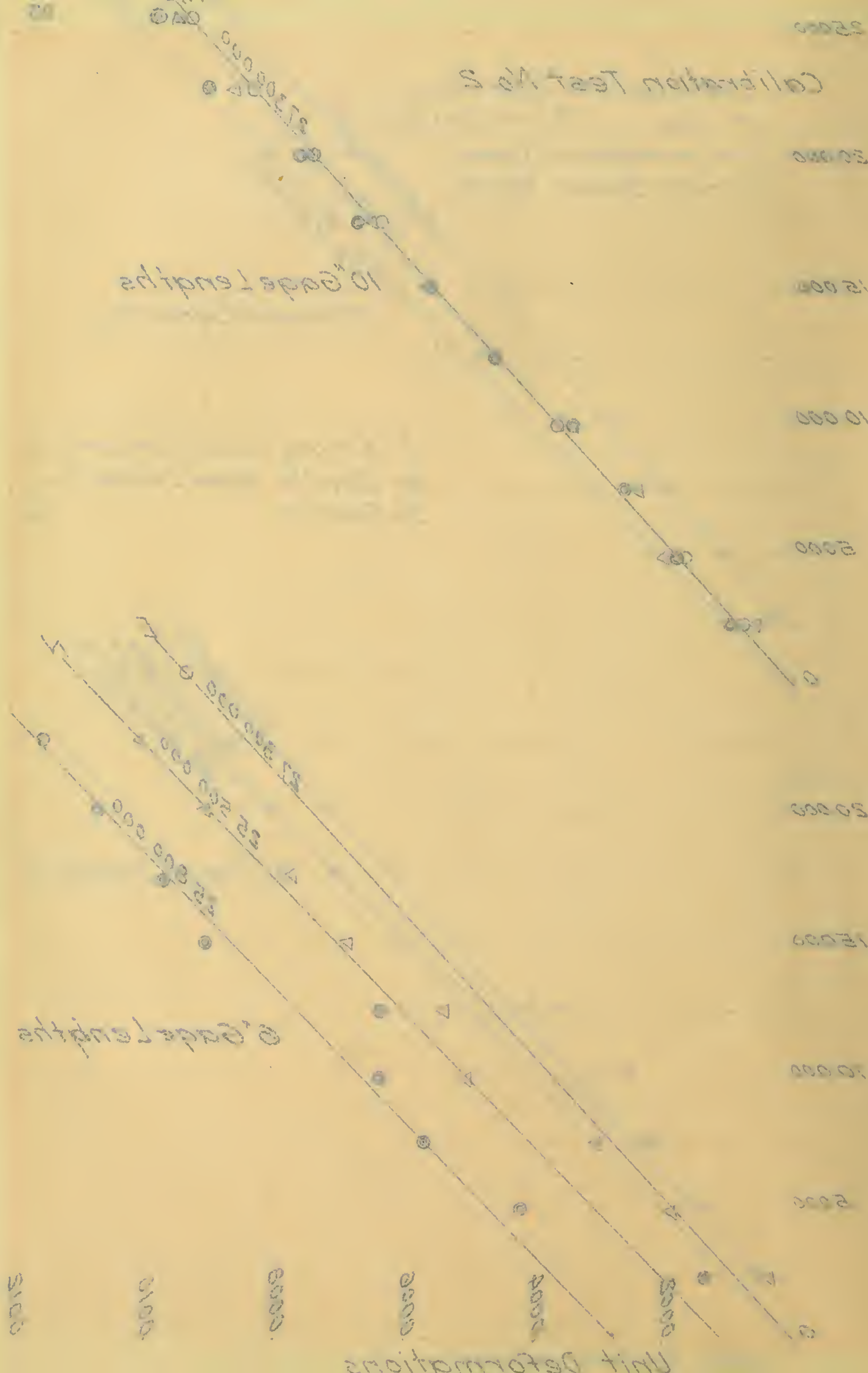
6" Gage Lengths
Unit Stress Indicated by Machine
Unit Deformations
27 300 000
27 300 000
25 500 000
25 800 000
N, E
E
N
S
25000
20 000
15 000
10 000
5000
0
20 000
15000
10 000
5000
0
.0002
.0004
.0006
.0008
.0010
.0012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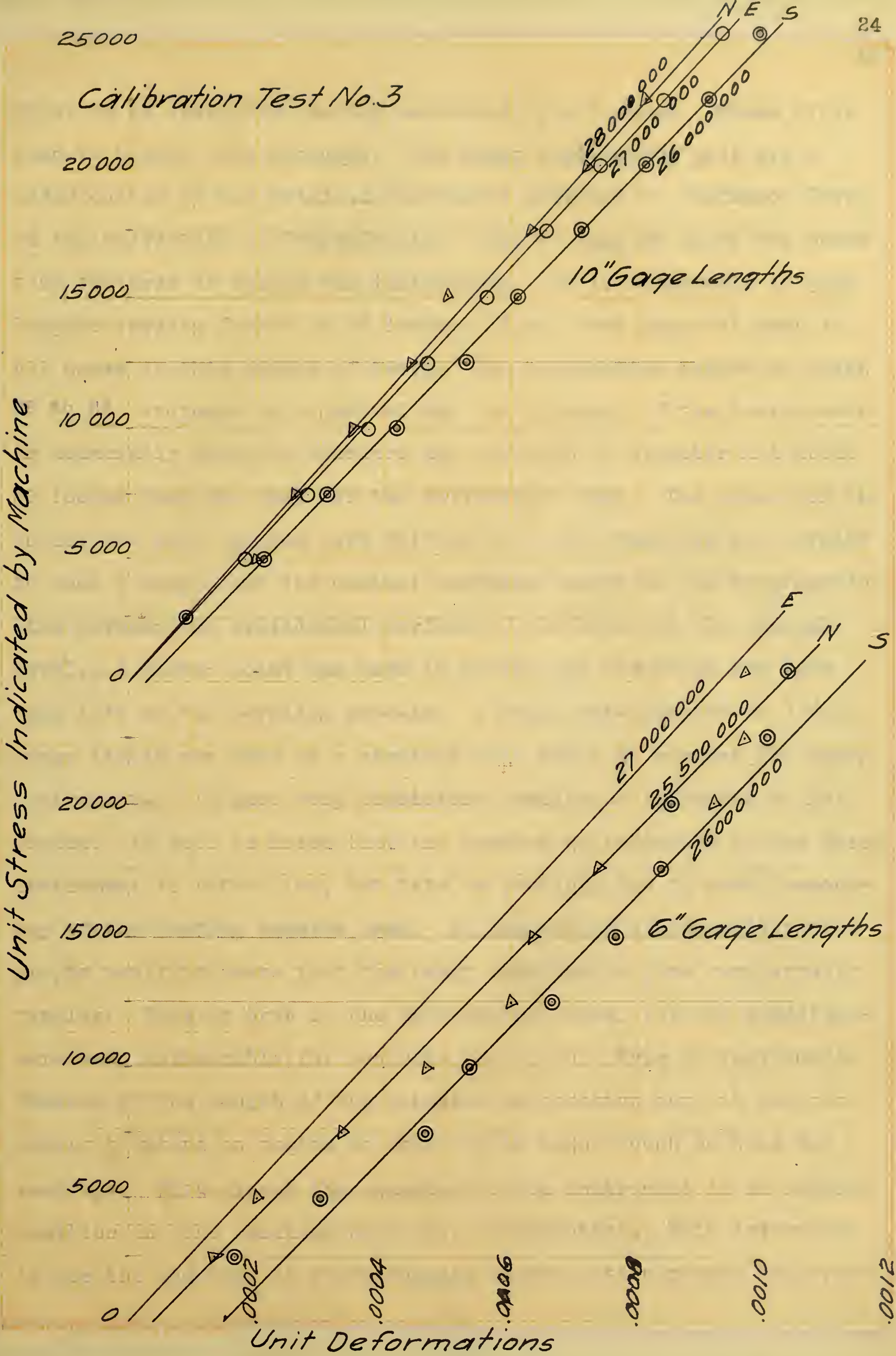
6" Gage Lengths
Unit Stress Indicated by Machine
Unit Deformations


Calibration Test No.3
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6" Gage Lengths
N
E
S
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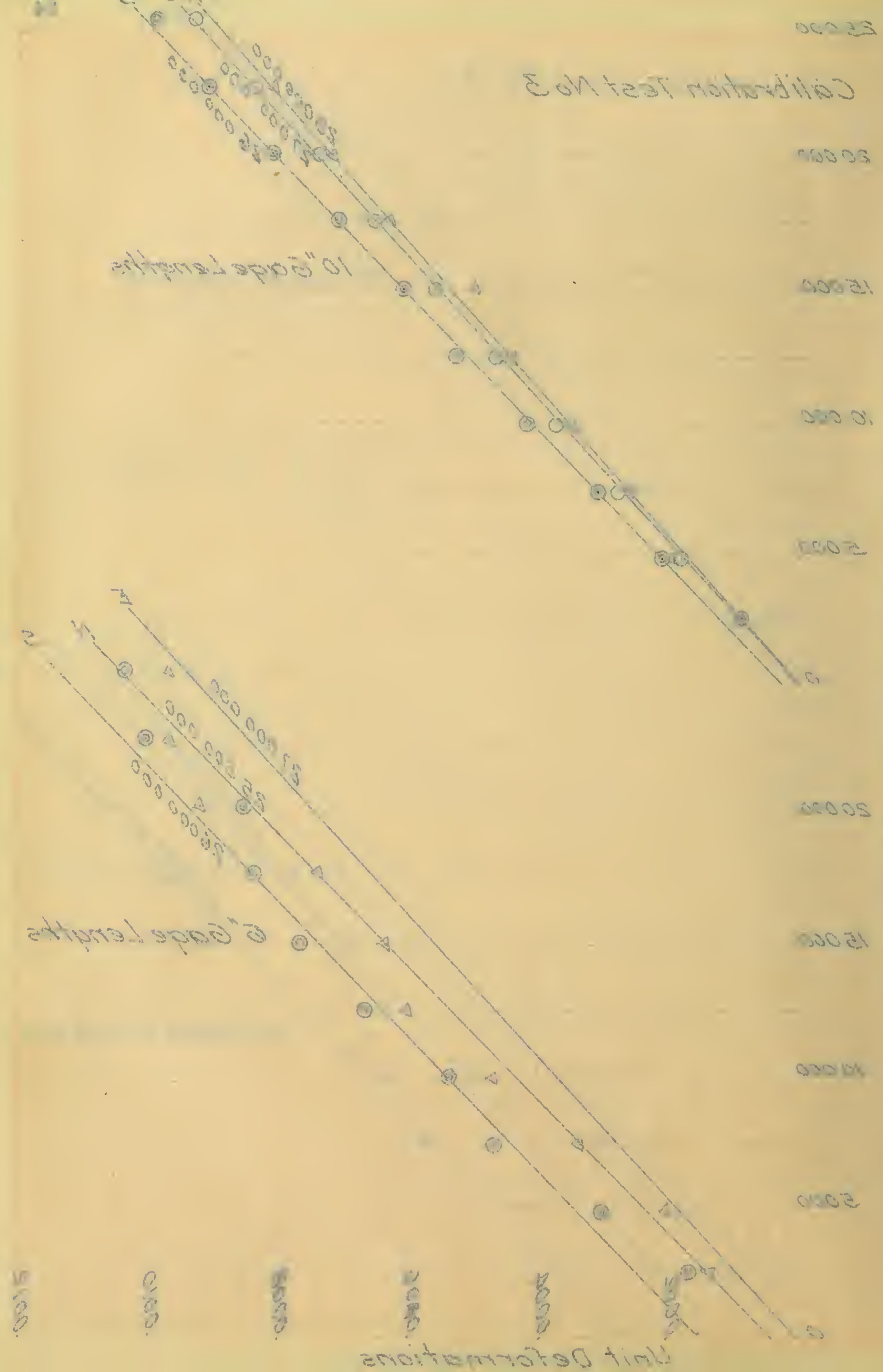
27 000 000
26 000 000
27 000 000
25 500 000
26 000 000
25000
20000
15000
10 000
5000
0
20000
15000
10000
5000
0
.0002
.0004
.0006
.0008
.0010
.0012
Unit Stress Indicated by Machine
Unit Deformations


Calibration Test No 3
10" Gage Lengths
6" Gage Lengths
Unit Stress Indicated by Machine
Unit Deformations

which is an assumption hardly warranted in all cases. These dials read to 0.0002 inch directly. The Berry instruments used are a modification of the original instrument invented by Professor Berry of the University of Pennsylvania. Fig. 3 page 20 shows the essential features of one of the instruments. It is adjustable to gage lengths varying from 6 to 10 inches. The 6 inch gage was used in all cases in this series of tests. The calibration curves on pages 22 to 24 indicate in a general way the accuracy of the instruments. An especially prepared standard bar one inch in diameter and about 30 inches long was used for the calibration test. The holes for the 10-in. and 6-in. gauges were drilled into the steel bar and beveled to such a slope that the conical surfaces rested on the intersection line between the cylindrical surface of the hole and the conical bevel. A wooden point was used to smooth any burr which may have been left by the beveling process. A Ewing extensometer of 8 inch gauge length was used as a standard with which to compare the Berry instruments. It gave very consistent results as indicated by the graphs. It will be noted that the modulus as indicated by the Ewing instrument is rather low, but this is probably due to some inaccuracy of the testing machine used. An inspection of the calibration graphs would indicate that the Berry instruments give very erratic results. This is true in the calibration tests, but the conditions were very unfavorable for accurate use of this type of instrument. Because of the length of the standard calibration bar, it was necessary to stand on chairs in order to be high enough to take the readings. This placed the operator of the instrument in an awkward position and the readings were very inconsistent. This instrument is one the accuracy of which depends almost entirely upon the oper-

ator, and in this awkward position the inaccuracies were worse than is believed to have been the case in the beam tests. A considerable pressure is necessary in applying the instrument to the gauge and if there is a vertical component either up or down, the reading will be incorrect. While it is believed that the results of measurements on the beams were more consistent, yet the results are valuable more as an indication of what was happening in a general way than as giving exact measurements of stress. A very good index as to the accuracy of the measurements on the horizontal steel may be had by an inspection of the plotted observed average unit stresses over the support and at the center of the spans of some of the beams as compared with the theoretical graphs accompanying the same. The results of measurements on beams 376.5 and 376.1 are not considered as accurate as those on the other beams, because it was our first experience in the use of these instruments. It is not believed that the drilled holes in these beams were prepared quite so well as for the later tests, since they were 5/64 inch in diameter, and of course, this greatly reduced the section in case of the 0.21 inch diam. steel used for web reinforcement in beams 376.1 and 376.2. Very great care was taken to eliminate every possible source of inaccuracy and it is believed the results are as good as could have been obtained under the conditions. The short gauge length used of course did not give as consistent results as a longer gauge length would have given, but it does give a better idea of the variation of the stress along a rod imbedded in concrete. The instrument magnified the deformations 5 times, indicated to 0.001 inch. This means an actual deformation of 1/5 this amount. In order to eliminate errors in observations due to change in the temperature of the

instrument, a standard bar was used on which the instrument was read before and after each series of measurements for each load. For beams 376.1 and 376.5 a naked steel bar was used, but for all others a bar imbedded in concrete was used in order to approach more nearly the temperature conditions surrounding the steel rods in the beams. In this way any change from the initial reading on the standard bar would indicate the amount of correction to be applied to the measurements on the steel in the beams.

In order to gain access to the steel reinforcement, for the purpose of drilling the contact holes, it was necessary to cut small holes into the concrete. The photographs, pages 102-115, show the holes, which are representative of the size of holes cut. When the steel was deep, of course it was necessary to cut larger holes than when it was near the surface, but it is not believed that these holes materially weakened the web or changed the conditions appreciably which would obtain in a similar beam without the holes. Of course, the cracks may be localized at these points somewhat, as is shown on some of the drawings, but this does not seem to be serious.

In order to detect the slipping of the unanchored bars, Ames dials were screwed to U-shaped wooden yokes which were clamped securely to the ends of the beams in such a way as to allow the plunger to press against the ends of the horizontal bars. An observer was stationed at each end of the beam and as soon as any slip was indicated the load was recorded. The dials read directly to 0.001 inch and by estimation to 0.0001 inch. The stirrups of beam 373.2 were the only ones which permitted the use of this arrangement for detecting slip, and a dial was placed against one leg of only one of the stirrups. It is believed that the indications on the Ames

dials thus used gave the true moment of slip of the rods. In order to see if this may have been caused by a relative motion of the wooden yoke and the beam, a dial was fastened to the yoke used on the west end of beam 372.2 and its plunger placed against the concrete. No such motion was indicated.

The cubes were tested in a 100 000 lb. Riehle vertical-screw testing machine with a speed of 0.05 inch per minute. Plaster of Paris was placed on both compression faces a day or more before testing in order to insure a uniform bearing. Cubes for beams 374.1, 375.1, 376.1, and 376.5 were tested with several layers of building paper between the plaster and the bearing plates of the machine. This arrangement gave a much lower strength as indicated by the table II page 30. A spherical bearing block was used in all cases. The cubes were tested at approximately the same age as the corresponding beam.

The control beams were tested in the same machine and at the same speed as the cubes. Third point loading was used over a total span of 3 feet.

IV EXPERIMENTAL DATA AND DISCUSSION

10. Notation Used.—The following notation will be used in discussing the results of the tests:

f_s = unit stress in steel;

f_c = unit stress in concrete;

E_s = modulus of elasticity of steel;

E_c = modulus of elasticity of concrete;

n = E_s/E_c;

T = total tension;

C = total compression;

M_s = moment of resistance relative to the steel;

M_c = moment of resistance relative to the concrete;

M = bending moment;

A = steel area;

b = breadth of beam;

d = net depth of beam;

k = ratio of depth of neutral axis to depth d;

j = ratio of lever-arm of resisting couple to depth d;

d' = jd = lever-arm of resisting couple;

p = steel ratio = A/bd;

o = circumference or periphery of one reinforcing bar;

m = number of reinforcing bars;

u = bond stress per sq. in. on the surface of the reinforcing bars;

v = vertical shearing and horizontal shearing stress per sq. in.

V = total vertical shear at any section.

TABLE II

CRUSHING STRENGTH OF 6-INCH CUBES

Beam No.	Crushing Load	Average Ultimate Unit Stress
371.2	93 000 93 450 89 300	2 550
372.1	68 100 82 450 68 860	2 030
372.2	84 850 89 430 76 500	2 230
373.1	72 500 82 970 74 470	2 130
373.2	93 800 101 000 + 95 000	2 680
374.1	60 380 59 600 57 640	1 630°
375.1	64 600 54 120 67 050	1 716°
376.1	53 100 55 000 56 420	1 520°
376.2	69 280 74 840 73 440	2 020
376.5	52 680 59 630 60 140	1 600°
376.6	97 750 84 860 89 790	2 510

° Load applied through paper cushions, giving low values.

TABLE III

TESTS OF 6 x 8 INCH CONTROL BEAMS

Beam No.	Breaking Load	Modulus of Rupture
371.2	3 230	303
372.1	3 600	338
372.2	3 390	318
373.1	2 605	244
373.1	3 300	310
374.1	2 700	254
375.1	4 700°	440
376.1	3 140	294
376.2	2 830	265
376.5	Broken before test	
376.6	4 650	436

° Machine ran on high speed during this test giving high value for modulus of rupture and breaking load.

TABLE IV

Values of j Used in the Calculations

Beam No.	j
371.2	0.87
372.1	0.84
372.2	0.84
373.1	0.84
373.2	0.84
374.1	0.85
375.1	0.86
376.1	0.86
376.2	0.86
376.5	0.87
376.6	0.87

The above values are for the regions of negative moment.

11. Explanation of Tables, Diagrams, Drawings and Photographs.—Indexes to tables, diagrams, drawings, and photographs will be found following the table of contents. The following explanations are believed to be sufficient to the understanding of them.

Tables.—Tables I and II are explained under the description of materials. Table III contains the modulus of rupture for the control beams as calculated by the common flexure formula $M = \frac{SI}{C}$. Table VI contains the amount of slipping of the unanchored rods at the loads indicated. If the progress of applying the load to any particular beam was not as indicated, the tabulated tension, bond, and slip have reference to the load nearest to the one at the heads of the respective columns. Under "gauge used" are tabulated the particular gauge lengths on which measurements were taken of the deformation of the steel. The index number refers to the number of the rod on which the slipping occurred. These numbers are indicated on the drawings of the several beams tested. The tension in the steel and the loads on the beam are indicated in thousands of pounds and tenths thereof. The amount of slipping is indicated in ten-thousandths of an inch. Table V is a summary of some of the measured and calculated data, as well as of the general properties of the test specimens. The per cent of web reinforcement is figured as follows: Suppose the vertical stirrups are spaced 4-in. apart. Then the per cent equals the cross-sectional area of the two legs of the stirrup divided by the area found by multiplying the spacing by the width of the beam. If there is also some inclined steel the per cent of it will be found in the same way, using as the spacing the horizontal distance between consecutive web members. The straight line distribution of stress was used as a basis of calculation for getting

the calculated values given in the table. The computed bond stresses were obtained by using the formula $u = \frac{V}{mo\,jd}$. Under the column headed "longitudinal reinforcement," the loads given are the loads on which the calculations of the steel stresses are based. All other computations are on the basis of the ultimate loads reached.

On pages 131 to 155 will be found the tabulated results of the measurements of stress made on each beam. The letters refer to the gauge lengths, the load has reference to the total load applied at four points on the beam. Opposite each load will be found the instrument reading corrected for temperature variations, and under the instrument reading will be found the deduced stress in pounds per sq. in.

Diagrams.—Shear — stress diagrams will be found on pages 70 to 84.. The values of the average unit shear in pounds per sq. in., are used for the ordinates and the unit stresses for abscissas. The letters refer to the gauge length, and a wavy line across a plotted point indicates the time at which a crack was observed to open across the gauge length. The moment — stress diagrams for some of the gauge lengths on longitudinal steel will be found on pages 85 to 93.. The moment has reference to the external bending moment at the section through the center of the gauge. Alongside of some of these plotted results are given the graphs representing the theoretical stresses on the steel, assuming beam action throughout, and using the straight line theory of stress distribution. On pages 94 to 100. are given the load-deformation diagrams for beams 374.1 and 375.1. The deformations were measured with the wire wound dials. The calibrations curves of the Berry instruments are given on pages 22 to 24. Three tests were made and two gauge lengths on the bar

were used for each instrument designated by N and S, meaning that one gauge length was on the north side of the bar and the other on the south side. During each test one instrument was set for a 6-in. gauge and the other one set for a 10-in. gauge.

Drawings of Beams.—At each increment of load applied to each beam, the visible cracks were traced over on the beam with a pencil and the limits of the cracks opening at any particular load were marked on the whitewashed surfaces. After the tests, one of the faces of each beam was carefully sketched, the steel being located accurately. Care was taken to sketch the cracks carefully since it was believed their location would affect the problem under investigation. These sketches are shown on pages 120 to 130. Plan and sectional views are also shown on the same sheets.

Photographs.—The photographs of the tested beams were taken after failure of the beams and after release of the loads. The figures on the beams indicate the loads at which the cracks opened up. In most cases the cracks were traced over with a lead pencil for the purpose of photographing. The notes accompanying the other photographs are self-explanatory.

12. Phenomena of Tests.—Before taking up a discussion of the results, a detailed account of the phenomena observed will be given in so far as this was observed during the test and determined by cutting up some of the beams and noting anything which would affect the results of the tests. The beams will be taken up in numerical order.

Beam No. 371.2.—This beam was first tested as an overhanging beam in the same way as the others. It failed over the supports due to the steel passing the yield point. The maximum load attained was

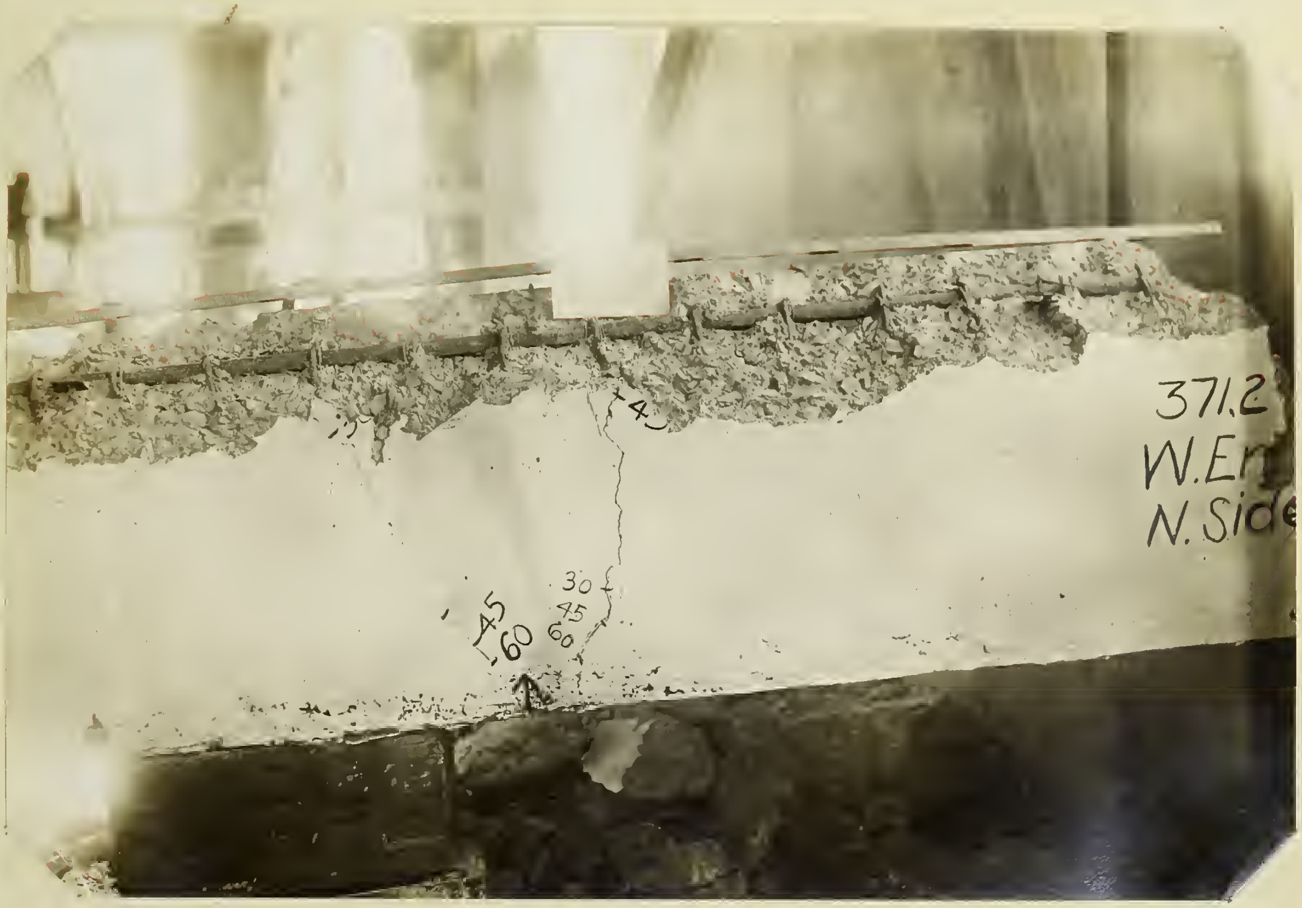


ILLUSTRATION OF TYPICAL SETTLEMENT CRACK.

SCALING OF ROD AT POINT IMMEDIATELY UNDER THE CARD.

65 400 lbs. After failure over the supports, the middle portion was tested as a simple beam as shown in Fig. 7 page 120. This portion carried a maximum load of 49 000 lbs. and failure was by tension in the horizontal steel in the center.

Slipping of Rods.—The concrete was cut away from the anchored ends of the longitudinal rods but no movement of the bars was visible to the naked eye.

Settlement Cracks.—Underneath the top horizontal rods very large openings were found due to the settlement of the concrete away from the rods while wet, The photograph on page 36 shows the crack under one of the rods at the west end of the beam, this crack extending the entire length of the horizontal portion of the rod over the support. The settlement cracks under the other rods were practically as serious. At the point where the top horizontal rods bent downwards, very serious settlement cracks were found, and just above the bend the steel had pulled away from the concrete leaving a small crack. This probably occurred when the high stress came upon the rod causing it to partially straighten out.

Under some of the stirrups inside the points of inflection, serious settlement cracks were found under the horizontal portion of them.

Necking of the Steel.—At the crack which formed about 6 or 7 inches west of the west support, the 3/4 inch steel rods were found scaled indicating that they had necked at this point. No scaling was found immediately over the support. At the east end the concrete was broken away with a sledge hammer which so battered the rods that no scaling could be detected.

Crushing of Concrete Between Stirrup and Rods.--No crushing could be detected between the stirrups and the horizontal 3/4-inch rods.

<u>Beam No. 372.1</u>.—This beam failed by tension in the steel at the center. The steel over the supports had also probably reached the yield point since the unit stress at a load of 100 000 lbs. was about 32 000 lbs. The crack which opened up due to the steel passing the yield point begun to open appreciably at the beginning of the 10 000 pound increment after a load of 100 000 pounds. The ultimate load reached was 110 000 pounds.

Slipping of the Rods.—As shown in Fig. 4 page 39 Ames dials were placed against the straight rod at the west end, and also against the rod anchored by a 180° bend. Slipping of the rod began at about the same time in the case of the anchored rod as for the straight rod. At the east end no instrument was placed against the anchored rod, but a crack indicated that slipping had occurred. No cracks were found indicating slip of the top horizontal bars which terminated near the inside load points.

Settlement Cracks.—In a number of places the aforementioned settlement cracks were found under the 3/4 inch rods, being as much as 0.1 inch wide in some cases.

Crushing of Concrete Under Bends.—The concrete was cut away from two of the bent down rods and a crack was found at the point indicated by A in Fig. page 48. Since the crack was local and no line of cleavage found, the concrete must have crushed underneath the bend, or else a settlement crack allowed the bend to straighten out somewhat. No settlement crack was observed although it may have been too small to be observed. Even if there was no such settlement

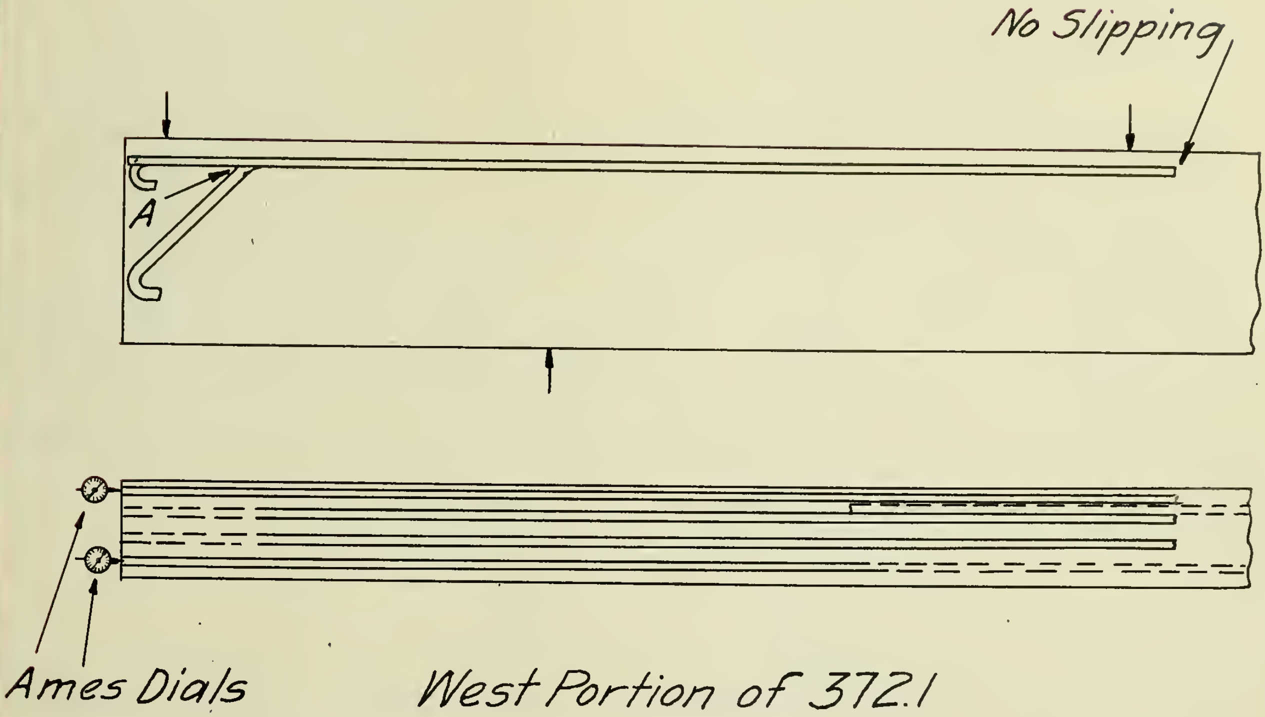


FIG. 4

crack, the concrete underneath was evidently less dense than elsewhere, due to settlement of the wet concrete. The adhesion of the mortar may have prevented the formation of a noticeable crack.

Crushing Between Stirrups and Horizontal Rods.—No crushing of the concrete between the stirrups and the horizontal bars was found.

Beam No. 372.2.—It is a little hard to say just what the cause of final failure was. The steel in the middle of the beam had passed the yield point, the steel over the supports had nearly reached the yield point, and measurements at gauge T indicate that that rod had passed the yield point. The crushing on either side of the beam and just under the bends in the longitudinal rods may have been the cause of final failure. The concrete at these points buckled outward. These pieces were removed and the concrete immediately under the bends was found crushed to a powder. This, together with the slipping of the unanchored rods caused the two diagonal cracks, shown in the figure, to open up considerably, the one about 12 inches from the west load point attaining a width of about 1/8 inch at latter stage of the loading. After release of the load, this crack partially closed until it was only about 1/16 inch wide. This would seem to indicate that the two unanchored rods had not reached the yield point opposite the gauge T.

When the load reached about 79 000 lbs., it was some time before the scale beam of the machine indicated an increase of load. The slowness with which the load came on would indicate that deformation somewhere was taking place very rapidly. At about this same load the slipping of the two unanchored 3/4 inch rods at the west end was 0.0012 inch for the north one and 0.0007 inch for the south

one. The large crack above mentioned begun to open appreciably at this load.

This beam was not cut into, hence no further data were obtained.

<u>Beam</u> <u>No</u>. <u>373.1</u>.—This one also failed by tension in the steel over the support. The horizontal rods were found scaled after breaking up the beam, and measurements were made to detect any necking of the rods, with the following results. Referring to the figure below it will be noted that the horizontal rods are numbered 1, 2, 3, 4, 5, 6, 7, and 8 for convenience in referring to them.

Plan View

Micrometer calipers reading to 0.001 inch were used to gauge the rods. Rod 1 had a diameter of from 0.752 to 0.755 at points of low stress, 0.744 at a point 25 inches from the end of the beam or about at the large crack in the beam, and 0.753 at a point 36 inches from the end or over the support. The rod was gauged every 2 inches. This indicates necking of the rod 25 inches from the end. Scaling of the rod at this place was also noticeable. Rod 2 showed a variety of diameters, hence all will be given herewith:

17	20	24	25	26	27	28	30	32	34	36	inches.
.746	.738	.746	.744	.743	.738	.741	.740	.747	.747	.745	inches.

The figures in the first line are the distances in inches from the end of the beam, and those of the second line are the measured diam-

eters of the rod at the respective points. Although this rod showed slight scaling and a brownish color at a point 26 inches from the end, the measurements do not indicate much necking. Rod 3 showed slight scaling and a brownish color at a point 26 inches from the end. The gauging of this rod are also given in full:

18	20	22	24	25	26	27	28	30	32	34	36 inches
.743	.737	.737	.741	.740	.735	.735	.734	.740	.743	.743	.742 "

Rod 5 showed a diameter + 0.752 to 0.753 at points of low stress, 0.750 at the large crack just over the letter T, and 0.751 over the support. Rod 8 had a diameter of 0.762 to 0.763 at points of low stress, 0.758 at the crack, and 0.760 over the support + 36 inches from the end. No necking could be detected on rods 4, 6, and 7. The crack at the top of the east end of the beam and 25 inches from the end opened up about 3/16 inch. There was no visible closing up of the crack upon release of the load. It will be noted from table VI page 118that the slipping of the unanchored 3/4 inch rods was very serious at a load of 100 000 lbs.

Crushing Under Stirrups.—The concrete was cut away from the stirrups carrying the highest stress. In two or three places it appeared that slight crushing of the concrete had occurred just at the point of bend in the stirrup where it passed over the horizontal bars. The evidence of crushing was not conclusive, however, as the concrete may have been loosened by the motion of the longitudinal rods at failure.

Slipping of Anchored Ends of Stirrups.— The concrete was cut away from the anchored ends of some of the stirrups. No slipping of nor crushing under the hooked ends could be detected.

Beam No. 373.2.—This appears to have been another tension failure of the horizontal steel at the support. The slipping of

the horizontal bars was not serious. In order to detect any slipping of the stirrups, an Ames dial was placed against the bottom of the north leg of the stirrup located 3 inches west of the support. At 80 000 lbs. a slip of 0.0003 was indicated but this did not increase as the load increased. This dial was fastened to an iron U-yoke which was clamped to the beam. The compression of the concrete at the support may have caused the movement of the dial plunger. This beam was not cut up in order to investigate the points mentioned in connection with the other beams.

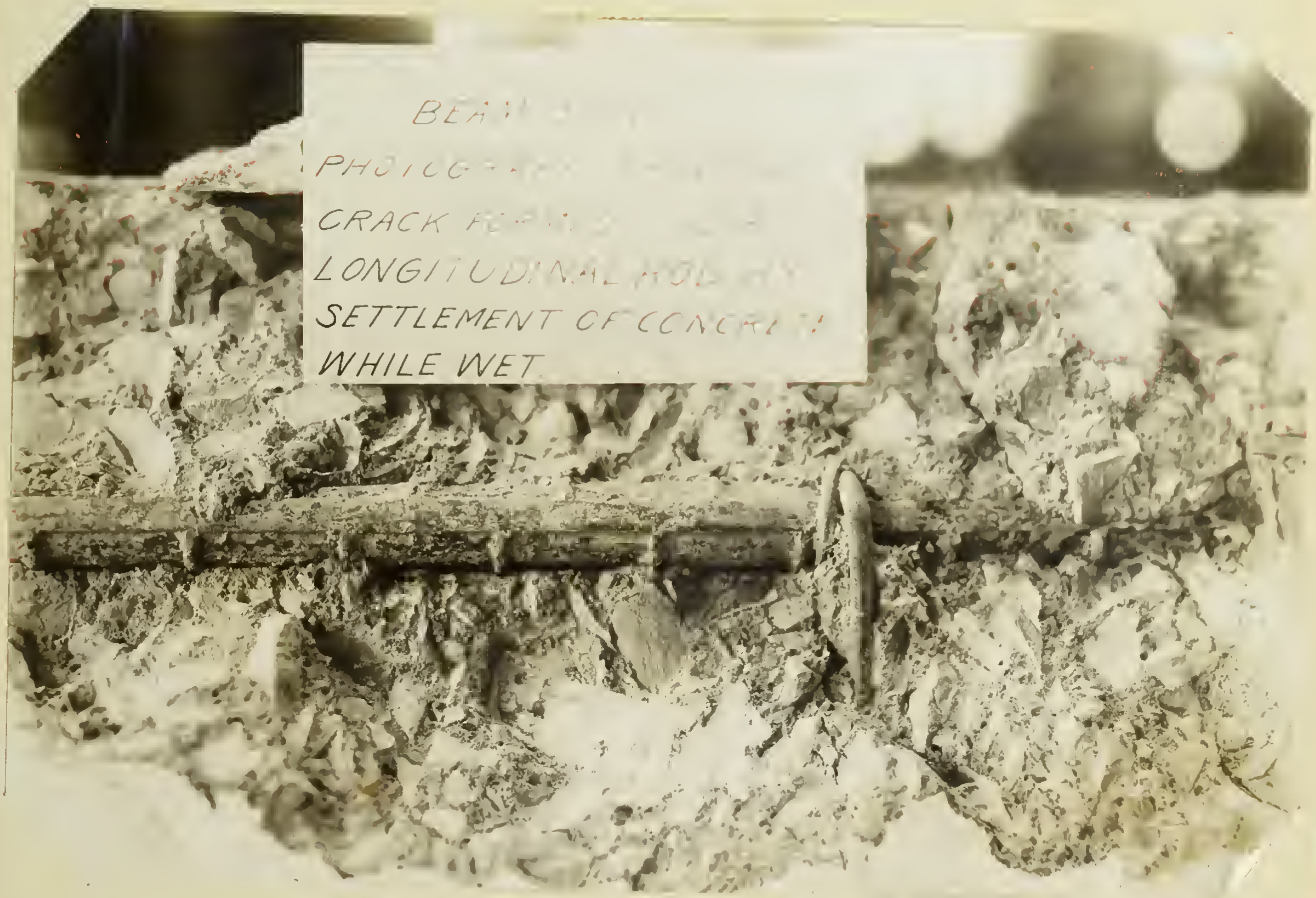
Beam No. 374.1.—This beam also failed by tension in the horizontal steel over the support, the unit stress as calculated = 39 200 lb. per sq. in. while the yield point of the steel was 35 100 lb. per sq. in.

Slipping of the Horizontal Rods.—At the east end of the beam the north Ames dial begun slipping at the beginning of the application of the load increment after 53 500 lb., but the amount of slip was not large. At the ultimate load of 57 100 lb. both Ames dials at the west end showed a slip of 0.0004 inch, the slip commencing at the beginning of the application of the last load increment. After release of the load the diagonal crack to the west of the support closed up almost entirely. After failure as an overhanging beam, the middle portion was tested on a 6-foot span with third point loading. This carried a maximum load of 61 000 lbs. The failure was by diagonal tension, the calculated stress in the steel at the center being 53 000 lbs.

Beam No. 375.1.—This beam failed at 102 100 lb. load, a comparatively large diagonal crack opening up outside the west and east support points.

Slipping of Rods.—After cutting away the concrete from the ends of the rods, the straight rods at the west end had slipped 3/16 inch and those at the east end had slipped 1/16 inch. At the west end the anchored ends of the bent down rods were found to have slipped slightly, leaving a crack at the end of the rod 1/16-in. wide, and at the point of bend 1/16 inch wide. No slipping of these anchored bent down rods was found to have occurred at the east end. Near the ultimate load the diagonal crack west of the west support, opened up and caused a large deflection of the beam. After release of load this crack did not close up which would seem to indicate that the bent down rods had passed the yield point caused by the stress having shifted from the unanchored rods, due to slipping. This being the case the failure would hardly be characterized as a diagonal tension failure. No instruments were used to detect the time at which slipping began.

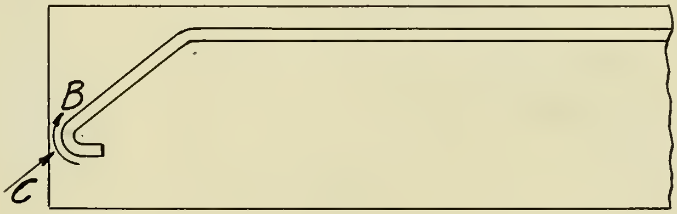
Beam No. 376.1—It is not known how to class the failure of this beam. The steel over the support did not reach the yield point, but the bent down anchored rods may have passed the yield point opposite the gauge S. The straight unanchored rods showed a slip of 0.0310 and 0.0450 inches respectively at the load of 122,300 lbs. This may have shifted the stress onto the anchored rods causing them to pass their yield point. As shown by the photograph, page 111a large diagonal crack opened west of the west support and did not close upon release of the load. It is further noticed that the concrete crushed at the bottom near the support and that the rod in the compression side buckled due to this compression. This crushing of the concrete and buckling of the rods occurred at the ultimate load. This all points to the belief that the failure was not primarily due to diagonal tension. It will be noticed that at the ultimate load of 141 100 lbs. a crack opened at the west end of


CRACK
LONGITUDINAL
SETTLEMENT OF
WHILE WET

the beam which was probably caused by the partial straightening out of the anchored rods at that point.

Beam No. 376.2.—The failure of this beam was similar to that of 376.1 although the ultimate load was much higher, reaching 178 000 lbs. The two inside rods over the support passed the yield point, but the outside straight ended rods did not reach the yield point. These unanchored rods at both ends of the beam showed a large amount of slip which probably caused the diagonal crack to open up so wide after the yield point of the two inside rods was passed.

Slipping of Rods.—At the west end of the beam, the 3/4 inch round rods which were anchored as shown in the figure, were examined after the test.



The north one was found to have slipped in the direction B about 0.02 inch, and in the direction C about 0.01 inch. The south one had slipped about the same amount. No slipping of the east ends of these same rods terminating near the inside load point was found. At the east end of the beam no slipping of the anchored ends of the bent down rods was found.

Settlement Cracks.—Numerous large openings were found underneath the horizontal tension rods over the supports. A rough estimate of the reduction of bond surface caused by the settlement would place it at about 25 per cent or 30 per cent of the total available bond area. No settlement cracks were found under the horizontal steel in the bottom of the beam, since these rested on

the concrete there as a means of support for the unit frame while pouring the concrete. Attention is called to the photograph page 45. Underneath three of the stirrups near the inside load point at the west end of the beam, the concrete was found to have settled away from the looped part of the stirrup near the top of the beam. This settlement crack was as much as 0.1 inch in some cases. This settlement was not found under the other web reinforcement similarly anchored.

Crushing Under Bends in Longitudinal Rods.—At the west end of the beam, cracks were found on the outside perimeter of the bend at the points where the two inside horizontal bars are bent downward. No cracks could be detected at these points of the corresponding rods at the east end of the beam.

Crushing of Concrete Under Stirrups.—At a few places near the supports, particularly the west one, the concrete beneath the web reinforcement and just over the horizontal rods could be scratched out with a small nail, seeming to indicate that the concrete there had crushed. This, however, was the case with only a few of the stirrups and the crushed condition may have been caused by other things than the stress in the web steel. The concrete was so badly broken up due to the collapse of the beam, that it was hard to tell just what caused this crushing under the stirrups.

The stirrup on which were located the gauge lengths O and P was found necked at the middle drilled contact hole.

Beam No. 376.5.—This one carried a load of 139 300 lbs. before failure. It is believed the slipping of the unanchored rods was the primary cause of failure, since the north bar at the west end slipped a total amount of 0.21 inch and the south one 0.52 inch. It will be noted from the photographs on page 114. that the con-

crete crushed and buckled between the west support and the inside load point. Slight crushing occurred around both supports on the north face, it appearing from the way the concrete flaked off that this crushing was in a horizontal direction. Slight crushing also occurred just west of the inside west load point, and just under it. All of this crushing was at a load of 139 800 lb. On account of the 18 inch H-Beam showing signs of crippling at a load of 139 800 lbs., the load was released, the steel beam strengthened, and the loading applied rather rapidly. Only 122 300 lbs. could be applied. At this load the crushing between the west support and the inside load point became very serious. Soon after this the buckling on the north face of the beam occurred.

Slipping of Rods.—The west ends of the unanchored 3/4 inch horizontal bars were too deep to easily find and apply instruments to. These unanchored rods at the east end were flush with the end surface of the beam, permitting the use of Ames dials for measuring slip. When the 139 800 lbs. load was taken off, the dials showed a backward movement though by no means enough to reach the initial zero reading. The amount of slipping of the west bars, as above mentioned, was found by cutting away the concrete. The bent down anchored rods as shown by sketch slipped slightly. At B the crack between the outside perimeter of the rod and the concrete was about 1/32 inch wide; and at C the crack could scarcely be detected by the naked eye, though it was apparent that some motion had occurred It may be mentioned in this connection that this is the only beam

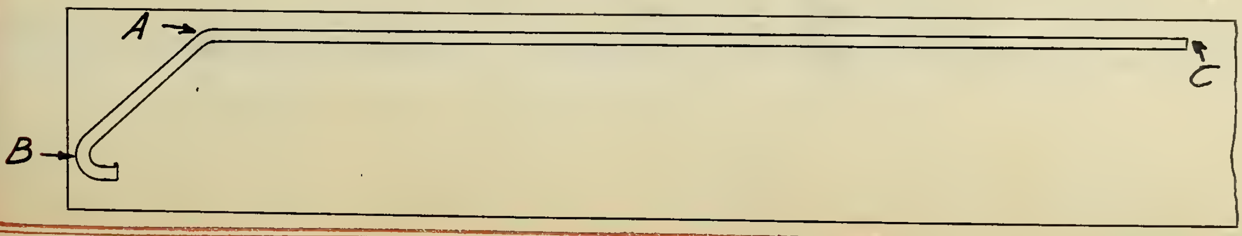


ILLUSTRATION OF CRUSHING OF CONCRETE IN FRONT OF CORRUGATIONS OF BARS

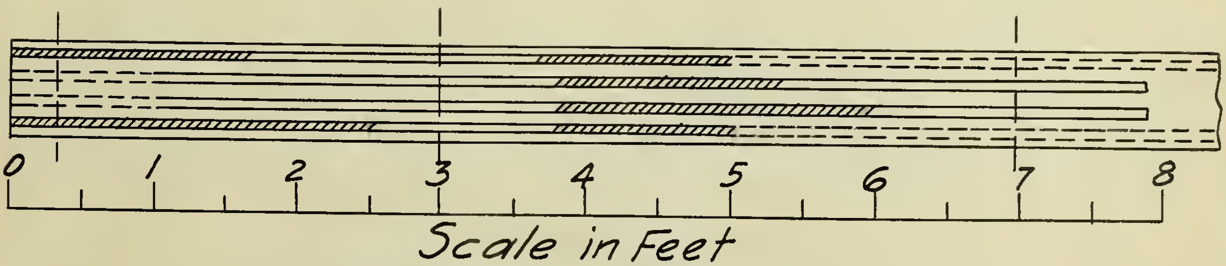
An examination of the above will show the places at which the crushing occurred. It should be studied in connection with the figure referred to above.

showing any slip of the bars at this point. In the case of the horizontal steel on which the gauge length P was located, at 139 800 lb. the rod slipped so that the concrete prevented putting the instrument point in for the purpose of taking a reading.

Crushing of Concrete Under Stirrups.—At the west end of the beam, the concrete was cut away and the concrete between the loops of the web steel and the horizontal rods was examined. In many places the concrete could be scratched out with a small nail, indicating that the concrete had been crushed, but this crushing may have been due to other causes at time of collapse.

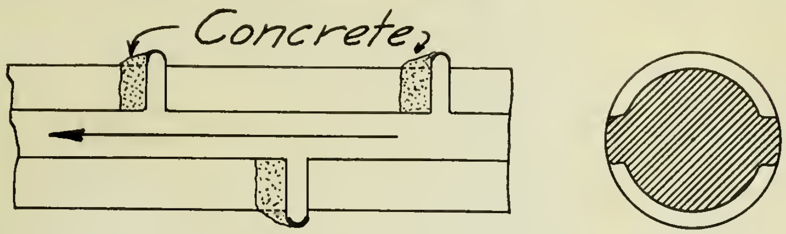
Beam No. 376.6—This failure was evidently by tension in the steel over the supports. In the latter stage of the test the load indicated by the machine would drop off rapidly, indicating a rapid elongation of the steel. The cracks which opened up were all comparatively small, the largest being only about 0.02 inch wide.

Slipping of Rods.—The concrete was cut away from the anchored ends of the 3/4-inch rods, but no cracks indicating movement could be found. The top layer of concrete was removed from the horizontal rods at the west end of the beam. In a number of places concrete clung to the corrugations indicating that the bond failure had allowed crushing of the concrete in front of the corrugations when the tension in the steel was sufficient. The sketch given herewith shows the range over which this crushing was found.



Plan View of West End

The shaded portions of the rods show the regions over which this crushing occurred and caused the concrete to cling to the rods. Of course it is possible for slight crushing to take place and yet not leave visible evidence of it. The sketch below shows the appearance



Sketch Showing How Crushed Concrete Clung to Rods.

of the rods with concrete clinging to the corrugations. The large arrow indicates the direction of "pull" of the rods relative to the concrete.

Crushing Under Web Steel.—There was no visible evidence of crushing of the concrete between the horizontal rods and the web reinforcement. While cutting off the concrete in search of such evidence, it was observed that the concrete was very hard and flinty.

13. <u>Analysis of Test Data</u>.—In discussing the results of the tests, it must be kept in mind that the unit deformations found in the web reinforcement are given as average values over the gaged lengths. There would seem to be no question that the stress in a stirrup is not uniform over a 6-in. gage length, but varies from point to point. This non-uniformity of stress may be even greater when a crack forms across the rod, because then the steel may even pass the yield point at the crack. Measurements made after the steel had passed its yield point would not give the average stress since it is evident that the deformation is not proportional to the stress after the yield point is passed. The stresses found in the horizontal rods over the support and at the center of the span of the beam may be considered the maximum stresses at these points. This is somewhat of an assumption regarding the stress in the steel over the support where the moment is changing rapidly along the length of the beam. The measured stresses on the web reinforcement are valuable more as a means of comparison of phenomena rather than as exact data of the actual stresses occurring.

In the analysis, the following order of discussion will be observed: A. Do the results show the present theory of stresses in web reinforcement as given in the introduction, to be correct? B. Is there a similarity in the stresses measured on companion beams having identical properties? C. If the present theory is incorrect, do the results indicate a law of action of the web stresses? D. After these points have been considered, other observed phenomena of a general nature will be discussed, particularly as they affect web stresses.

A study of the graphs and tabulated test data will throw light on the stresses in the web reinforcement. It will be observed that there is very little uniformity in the results, but it is also clear that if the stresses found are any where near the maximum stresses present, the calculation of stresses by the formula $\frac{V}{jd}.s$ and $0.7\frac{V}{jd}.s$ gives values too high. For beams having only vertical stirrups 1/4-in. in diameter and spaced 4-in. apart, the unit stress in each stirrup for any load of P lbs. for the beams tested, will equal approximately 0.5 P according to the above formula, and assuming j to be 0.86, it is certain that not all the stirrups showed the same stress. Another noticeable phenomenon, as shown by the graphs, is the rapid increase in stress after the opening of a crack across the steel. It will be noticed that this rapid increase of stress often begun somewhat before the crack was visible, but this does not mean that the concrete web had not failed in tension then, because such failure may take place and the line of failure be invisible. Not only is there a marked difference among the stresses in various parts of similar web reinforcement, but very few of the measured stresses reached as high a value as 0.5 P. In view of the foregoing wide disagreement between observed and calculated stresses, and assuming the observed stresses to be approximately representative of the actual stresses, the conclusion that the calculated stresses are incorrect seems justifiable.

B

The next logical step is to seek some clue to the

A Comparison of Measured Web Stresses in Companion Beams.

	376.1								376.2						
Load=	15	30	45	60	80	100	120		15	30	45	60	80	100	120
Gage Length								Corresponding Gage Length							
Q	1.2	0.9	3.3	4.5	4.1	7.4	9.8	Q	c0.5	c3.0	c4.5	c3.1	c4.1	3.7	6.2
R	c1.0	1.6	5.4	7.8	14.8	19.1	22.4	R	c1.1	0.6	0.4	3.9	12.6	28.9	42.3
O	0.7	0.1	7.1	14.3	23.4	30.9	38.3	O	c2.1	c0.9	c4.0	c0.7	1.7	7.0	15.0
P	0.5	3.5	8.4	13.8	24.1	30.1	36.1	P	c1.0	0.1	1.2	3.1	8.8	14.9	22.5
K	c0.4	c0.4	c0.4	3.1	12.1	17.1	27.0	K	c1.8	0.1	0.6	5.2	12.3	24.1	37.3
I	0.4	0.8	6.6	13.8	20.4	32.4	51.4	I	c6.3	c5.1	c4.4	c4.6	c3.7	c5.4c	5.5
J	c0.9	2.9	6.9	18.8	28.1	39.8	57.8	J	c1.8	c1.4	c3.3	0.9	4.8	7.9	9.4
H	0.3	0.7	11.0	18.2	25.8	31.1	16.8	H	c1.5	2.9	1.3	2.2	5.8	4.1	6.3
D	1.1	1.5	3.6	9.1	2.4	17.4	21.1	D	c0.1	c1.0	0.1	0.7	5.7	11.0	17.4
C	c1.0	c0.6	c0.2	2.0	3.0	4.0	3.3	C	c3.0	c5.3	c4.6	c0.4	4.8	10.8	10.6

By corresponding gage length is meant a gage length located at approximately the same relative point on beam 376.2 as the opposite gage length listed under beam 376.1

All loads and stresses are expressed in thousands of pounds. c= compression, otherwise the stress is tension.

A Comparison of Measured Web Stresses in Companion Beams.

	372.1						372.2				
Load =	30	45	60	80	100	Load =	30	45	60	80	100
Gage Length						Corresponding Gage Length					
O	1.8	5.5	5.9	18.7	29.2	O	c1.0	0.0	2.0	11.0	Beyond yield point
N	c4.9	3.7	c0.6	c0.8	15.7	N	c2.0	c3.0	3.0	16.0	" " "
L	1.5	5.5	3.3	4.8	8.8	L	0.0	c1.0	0.0	3.0	23.0
M	c1.5	c4.0	c3.2	c1.7	c2.9	M	c1.5	c1.0	c2.0	c2.0	c 2.0
F	c0.1	2.4	c0.1	c0.5	o.7	F	1.0	1.0	3.0	10.0	15.0
G	0.0	0.2	0.6	2.0	4.8	G	0.0	0.5	1.5	0.5	2.5
D	0.0	0.0	c0.5	17.3	35.3	D	1.0	1.5	2.5	1.5	1.5
E	3.0	3.2	4.7	5.0	8.0	E	0.0	1.0	3.0	3.0	3.0
C	2.2	2.3	6.3	13.8	24.0	C	0.0	1.0	2.0	0.0	3.0

A Comparison of Measured Web Stresses in Companion Beams

			376.5									376.6			
Load =	15	30	45	60	80	100	120		15	30	45	60	80	100	120
Gage Length								Corresponding Gage Length							
O	1.9	c6.9	3.0	2.9	4.0	c0.2	5.2	O	3.2	c1.5	5.2	7.0	11.0	3.7	8.9
M	1.4	c4.8	1.0	3.2	6.0	4.6	7.5	M	0.8	1.8	4.0	7.3	12.0	18.7	23.6
K	1.7	c2.8	4.4	6.8	13.8	17.5	25.5	K	0.5	6.9	4.3	8.8	10.4	17.9	24.9
J	c0.2	c7.8	2.5	2.8	5.4	6.0	10.7	J	1.8	1.5	2.7	4.3	5.5	9.2	12.9
H	c3.4	c0.3	0.8	3.6	2.4	6.9	4.8	H	4.9	4.5	5.1	7.7	5.4	10.2	8.8
F	12.9	8.7	7.6	3.8	8.7	11.7	16.1	F	7.5	6.9	6.5	3.8	0.1	4.3	0.2
E	13.0	10.2	2.6	6.3	4.8	4.9	2.4	E	0.8	c3.2	c3.1	0.6	c1.3	c 3.3	c4.1
D	c6.5	c6.6	c5.7	c12.2	c9.8	c9.6	c10.6	D	2.8	3.0	2.6	4.0	3.4	c 1.0	3.6
C	15.0	14.1	12.5	10.5	11.2	12.7	6.5	C	1.5	0.8	2.6	c0.7	2.4	1.7	2.9

A Comparison of Measured Web Stresses in Companion Beams

	373.1							373.2					
Load= Gage Length	15	30	45	60	80	100	Load = Corres- ponding Gage Length	15	30	45	60	80	1000
T	4.1	5.0	5.0	8.1	8.1	8.6	T			2.0	5.0	18.4	28.4
Q			3.1	9.3	12.5	19.1	Q			4.7	c0.3	6.0	13.3
R			0.4	3.2	1.2	4.7	R			c2.8	c2.8	c1.1	2.5
K			0.7	4.5	3.5	3.3	K			1.3	0.3	3.0	4.0
L			3.7	6.5	7.2	6.7	L			0.7	3.0	7.0	15.0
I			4.2	11.2	17.6	24.2	I			0.3	4.3	9.3	10.0
J			6.8	7.9	14.6	21.9	J			6.1	9.6	19.3	25.6
H			4.7	6.7	12.2	12.2	H			0.7	2.4	2.0	1.4
D			7.0	8.1	10.1	12.3	D			0.0	0.7	0.0	2.0
B			1.5	1.3	2.3	5.1	B			1.0	0.3	0.0	2.2

conditions governing the web stresses. It is natural to expect gage lengths having corresponding locations on companion beams to show approximately similar stresses under the same load. To illustrate, consider two beams of practically identical properties and a stirrup in each beam 8-in. from the support. Then if measurements are made on the top 6-in. of these stirrups, one would expect to find about the same stresses at corresponding loads. As indicated by the tabulated test results on pages 54 57 arranged for comparison, it is evident even upon a casual study of them that there is very little similarity, either in magnitude or in the nature of the variation of the stresses as the load was increased. One factor likely to cause this is the quality of the concrete; another is the afore-mentioned caution that the measured stresses are not maximum stresses; another is the distribution of diagonal tension cracks.

C

Before taking up a study of the beams containing web reinforcement, it will be well to examine the action of a beam without web reinforcement and particularly the action of the middle portion of 374.1 tested as a simple beam. Referring to the sketch of it on page 60 it will be seen that a diagonal tension crack opened at a load of 34 000 lbs. which extended from the horizontal steel to within about 3-in. of the top of the beam. This crack was inclined at an angle of about 45° with the horizontal and if extended would terminate near the load point. On the other end the diagonal crack opened, making

approximately the same angle with the horizontal, but extending toward a point within the load point. Referring to Fig. page 60 which is a reproduction of the sketch of this beam, it may be considered that there is something in the nature of truss action after the diagonal crack has opened. Assume the portion to the right of the crack to be hinged at the point A and for simplicity assume that all the shear is carried by the concrete at the point A. This latter assumption is justifiable in the case of longer beams with third point loading where there is a considerable distance between the support and the bottom of the crack, because in that case the bars will strip off from the concrete at high loads and all the shear will be thrown on the concrete at A. Taking moments about B we have

$$13 H - 19 V = 0 \text{ but } V = \frac{P}{2}$$

hence

$$H = \frac{9.5 P}{13} = .73 P \text{ or about } \frac{3 P}{4}$$

On this basis when P = 61 000 lbs. which was the ultimate load, V = 35 500 lb. and the unit shear on the 3 X 8-in. section at A = 1 480 lb. per sq. in. also H = 45 750 lb. and the unit compression equals 1 900 lb. per sq. in. The compressive and shearing stresses at A are high and the failure was probably due to the former since it is probable that the rods at the bottom carried a large portion of the vertical shear. The actual compressive area at A may have been smaller, and the centroid may have been nearer the load point. Both of these factors would raise the unit compressive stress at A and failure might follow by crushing. The short distance between the load and the support of this beam would prevent a failure by stripping

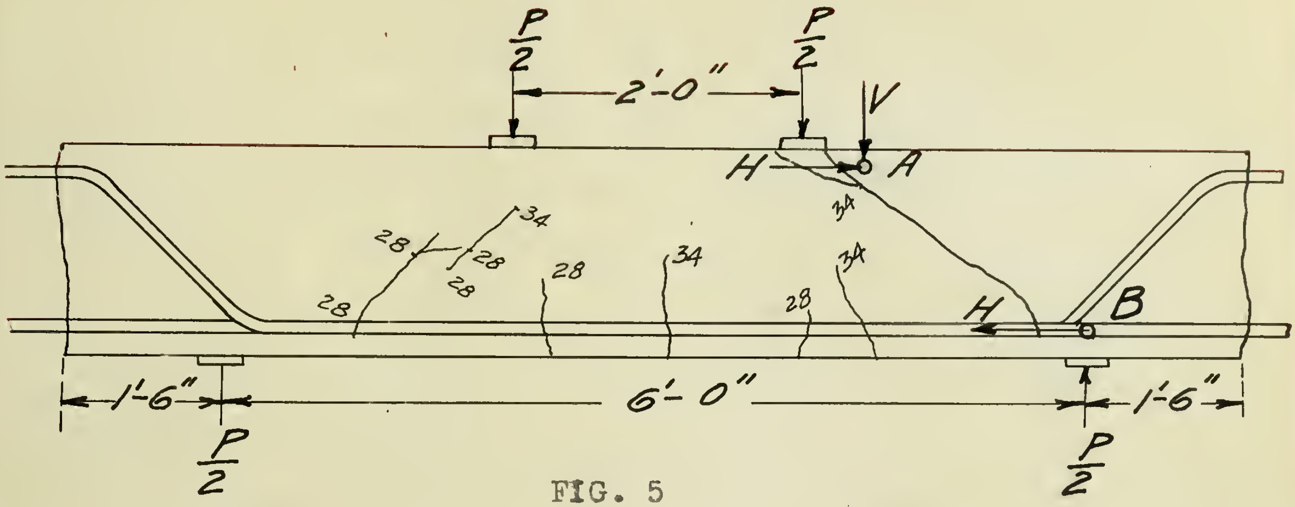

P/2
P/2
2'-0"
V
H
A
34
34
28
28
28
28
28
34
34
28
H
B
1'-6"
6'-0"
1'-6"
P/2
P/2

FIG. 5

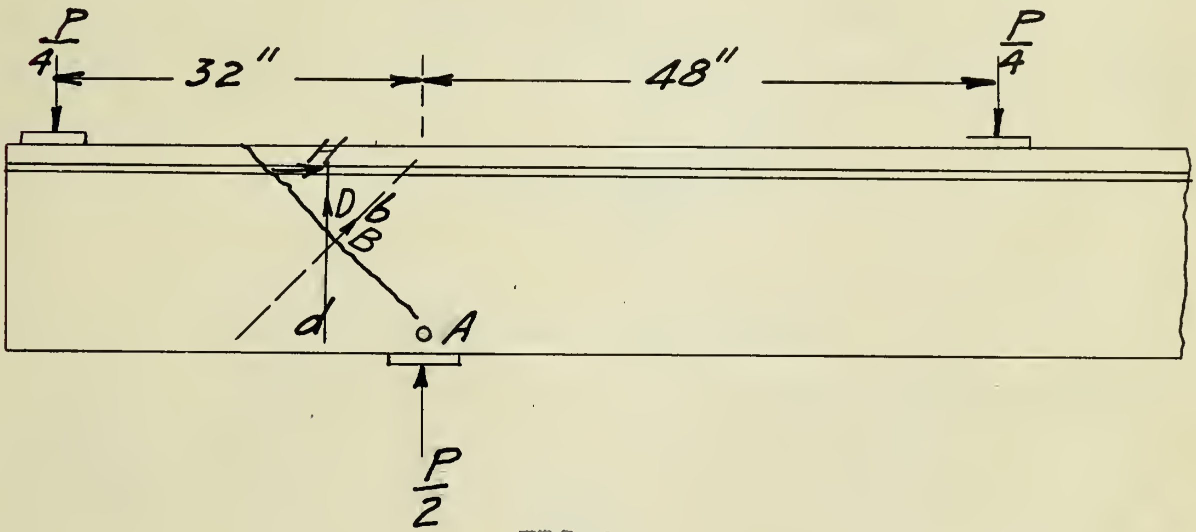

P/4
32"
48"
P/4
H
D
b
B
d
A
P/2

FIG.6

off the bars and this condition probably accounts for the high load carried by this beam. The bond stress developed on the portions of the rods over the supports was probably comparatively high, especially on the two running straight through. The stress in each rod at the crack equaled about 11 400 lb. and the area imbedded was about 52 sq. in. This would mean a bond stress equal to 220 lb. per sq. in. which was probably not enough to cause slipping.

Referring to Fig. 6 page 60 imagine a single stirrup across the typical diagonal tension crack and 8-in. from the support. Taking moments about A and again assuming no shear to be taken by the rods at H, there results,

$$13\,H + 8\,D = \frac{32\,P}{4}$$

$$\text{but } \frac{P}{4} = V$$

hence $D = \dfrac{32\,V - 13\,H}{8}$ = total stress in d.

From the conditions the amount of the stress in a is indeterminate. Before the crack opens very little stress will be thrown on d, but upon the opening of crack, the deformation in a vertical direction will throw some stress upon d and this stress will increase as the deformation in a vertical direction increases. If instead of the vertical stirrup at this section there is a rod inclined at 45°, then the stress on it will be

$$b = \frac{32\,V - 13\,H}{10.5}$$

This does not mean that the stress in b is less than in d. The amount of stress will depend upon the amount of deformation in the direction of the length of the web steel. The question will likely be raised here as to the relative deformation in the two directions. It is evident that before the crack opens

the stress on b will be much more than the stress on d since the maximum diagonal tensile stress acts approximately in the direction of the former, causing some stress in b from the beginning, although not very much on account of the strength of the concrete web. The amount of deformation in the direction of b will be the resultant of the amounts in horizontal and vertical directions. This would indicate that the deformation, and consequently the stress, in the direction b to be greater than in d. In a general way the results of the tests show this.

Had sufficient measurements been made on the horizontal rods at the points where the typical diagonal cracks opened, the total amount of the moment carried by the web reinforcement across the crack could be determined approximately. The measurements which were taken indicate in a general way the amount of stress taken by the horizontal rods at the crack. Referring to Fig. 8 page 121 of beam 372.1 it will be seen that the average unit stress indicated by gage lengths P and Q are fairly representative of the average stress in the four rods. At a load of 80 000 lb. the average stress measured over the typical diagonal crack to the west of the support is about 23 000 lb.

Taking moments about a point 2-in. above the support there results,

20 000 X 32 = 1.76 X 23 000 X 13 + (the sum of the moments due to the stresses in the stirrups)

This leaves a moment of 114 000 inch lb. to be taken by the stirrups, whereas the measured stresses in the stirrups give a moment of only 71 300 inch lb. This discrepancy is to be ex-

pected since what has been said heretofore regarding the measured stresses as being less than the maximum stresses present.

The moment taken by the web reinforcement will reduce the stress in the horizontal rods at the crack and this is found to have been the case. Were there no web reinforcement, and neglecting the portion of the shear carried by the vertical tension in the concrete surrounding the horizontal rods, the stress in these rods at the crack would equal the stress immediately over the support. This has reference of course to high loads,meaning by this term a stage of the loading after the transition from beam action to truss action.

D.

High Bond Stresses. - When the typical diagonal crack opens across the horizontal rods, as shown by the sketches, the stress in these rods at this point is much higher than would be the case were the stress there due only to beam action. Reference to (C) above will show this to be true. If there were no truss action, the stress at any point would be proportional to the external bending moment at that point, and this is the usual assumption used in figuring the stress in the horizontal steel. On this basis, the calculated bond stresses assumed to be developed in the beams tested would probably not be excessive. But with the high stresses found in the horizontal steel at these points of the beams tested, excessive slipping is to be expected. This fact shows the necessity of a longer length of rod for the

purpose of anchorage.

Slipping of Steel and Web Stresses. - If all the horizontal rods ran straight to the end of the beam, and all began slipping at the same time, the crack would open more, resulting in both horizontal and vertical deformation at the crack. This would throw more stress upon the web reinforcement, especially the inclined steel. If a part of the horizontal rods are anchored, a part of the stress will be shifted from the unanchored rods to these. The deformation will hence be greater and a higher stress will be thrown upon the web reinforcement than would be the case if all the rods were anchored equally. This would suggest the advisability of keeping the percentage of steel practically uniform throughout the length of the beam where the shear is constant, not only to prevent excessive web stresses, but also to prevent failure in the horizontal steel at the crack. Another effect of the slipping of the horizontal steel would be the crushing or shearing of the concrete at the bottom of the diagonal crack. This would be due to the reduction of the compression and shearing area caused by the crack opening up further down when the horizontal rods in the top of the beam slip.

Anchorage of Web Reinforcement to Horizontal Rods. - If the web reinforcement is inclined, it will be necessary to have rigid attachment, since the limit of the strength of an inclined member not rigidly attached would be the tensile strength of the concrete at the point of connection with the horizontal steel. Simply looping vertical stirrups under or over the horizontal steel, depending on whether the moment is

positive or negative at that point, would seem to be sufficient anchorage provided sharp bends are avoided and low compressive stresses between the stirrup and the horizontal steel are maintained. The restraining action of the surrounding concrete will permit higher working compressive stresses, and the use of small rods rather than the same cross section of larger rods will keep the compressive stresses low.

Deformed vs. Smooth Rods in Preventing Slipping. - An inspection of table VI page 118 will indicate little difference between the bond unit stress at the first slipping of the corrugated rods and the plain rods. In general it appears that the rate of slipping after the initial bond failure was greater for the smooth rods. The results are not conclusive, however, since the amount of settlement under the rods will affect the problem, and the amount of settlement is an uncertain quantity. But it is reasonable to expect the deformed bars to give higher bond resistance after the initial slip than the plain rods. The crushing of the concrete in front of the corrugations was very evident in all the beams having corrugated bars. This would suggest the advisability of having the corrugations so arranged that the crushing area in front of the corrugations would be greater. The anchorage would then be due both to the crushing resistance of the concrete in front of the bars, and the "running friction" on the horizontal surface of the bars.

Bending Horizontal Rods. - The crushing found under the bends of many of the longitudinal rods emphasizes a necessary precaution in this respect.. The crushing stress brought on

the concrete under the bend will be the component of the tension in the steel there which acts normal to the concrete surface under the bend. There will be components acting in nearly all directions due to the curved semi-cylindrical surface in contact with the rod. Only those acting in a vertical plane need be considered if the distance of this bend from the exterior surface of the beam is sufficient toprevent the outward buckling of the concrete noticed on beam 372.2. The use of bends of larger radius is to be recommended.

Settlement Cracks. - The settlement cracks found under the rods of several of the beams will show the necessity of providing additional length of bar for anchorage to take care of this. A search for similar cracks was not made in the cases of the other beams, but it is believed such cracks were there. In making the beams, the top horizontal rods were supported until the concrete was stiff, and the shrinkage and settlement in a depth of 15-in. would be serious in any case. It would not be safe to count upon more than about 1/2 of the total area of these rods as effective against initial slip. The gripping effect of the concrete on the rods will be greatly reduced if these cracks form. If the rods are deformed, and the settlement cracks are not too serious, a greater percentage of the total area would probably be available, though the uncertainty of this would not warrant one in counting on a greater percentage of the total surface of the rod.

After small cracks open across the horizontal steel, water will likely find its way into the open space under the rods

and corrosion might ensue. This is entirely possible since the cracks open at working loads, and in girders exposed to the weather the chance of water entering these cracks is good indeed.

Effect of Quality of Concrete. - If the concrete is of a good quality and well placed, the diagonal cracks will open at higher loads, the bond resistance will be higher, the crushing and shearing strength of the concrete will be greater and the deformations in the directions of the web steel will be smaller. The failures of beams 376.1 and 376.5 are believed to have been due to crushing of the concrete. The companion beams 376.2 and 376.6 carried a much higher load and failed by tension in the steel. It will be noted that in the former beams, the slipping of the horizontal rods was excessive, whereas in the latter, the slipping was small. Furthermore, as noted elsewhere, the concrete of beams 376.2 and 376.6 proved to be very hard as found during the process of cutting them up.

Deep vs. Shallow Beams. - The high loads as measured by the unit shear carried by the middle portions of beam 371.2 and 374.1 indicate that a relation exists between the depth of beam and the unit shear developed. The slenderness ratio may also affect the load that can be carried by beam of given cross section. These phases will not be gone into further but are mentioned as being fruitful subjects for investigation.

Horizontal Steel in Two Planes. - If the horizontal steel is placed in two planes at the points where the diagonal cracks are expected to open, greater stiffness will be secured and more of the total vertical shear can be counted upon as be-

ing carried by them. This will relieve the stress on the web steel as well as the crushing and shearing stresses at the point where the compression member of the truss is imagined as hinged.

Previous So-Called Diagonal Tension Failures. - In the light of this series of tests and the above discussion, it would seem that in many of the published records of tests in which the failures were classed as caused by diagonal tension, the statement of the method of failure may be questioned. Little or no attention has heretofore been given to the slipping of the horizontal rods, and little or no mention has been made of the stiffness of the horizontal rods affecting seriously the results of tests on beam, especially those without web reinforcement. Several of the beams tested by Messrs. Haeffner and Brooks this year showed serious slipping of the unanchored horizontal rods. These failures were classed as diagonal tension failures. Only four of such beams were examined, but each of these showed that serious slipping had occurred.

Tensile Stresses in Rods in Regions of Zero Moment.- Reference to the measurements on gage lengths E and F of beam 376.1 will show the largest unit stresses measured were approximately 27 000 lb. and 22 000 lb. respectively. These gages are near the point of inflection. Gage length V of beam 372.2 was on a point of one of the horizontal rods almost directly over the point of inflection. The largest unit tensile stress measured on this gage length was 17 500 lb.

Stresses in Web Steel beyond the Yield Point. - Reference to the graphs and the tabulated test data will show that the indicated average unit stress was greater than the

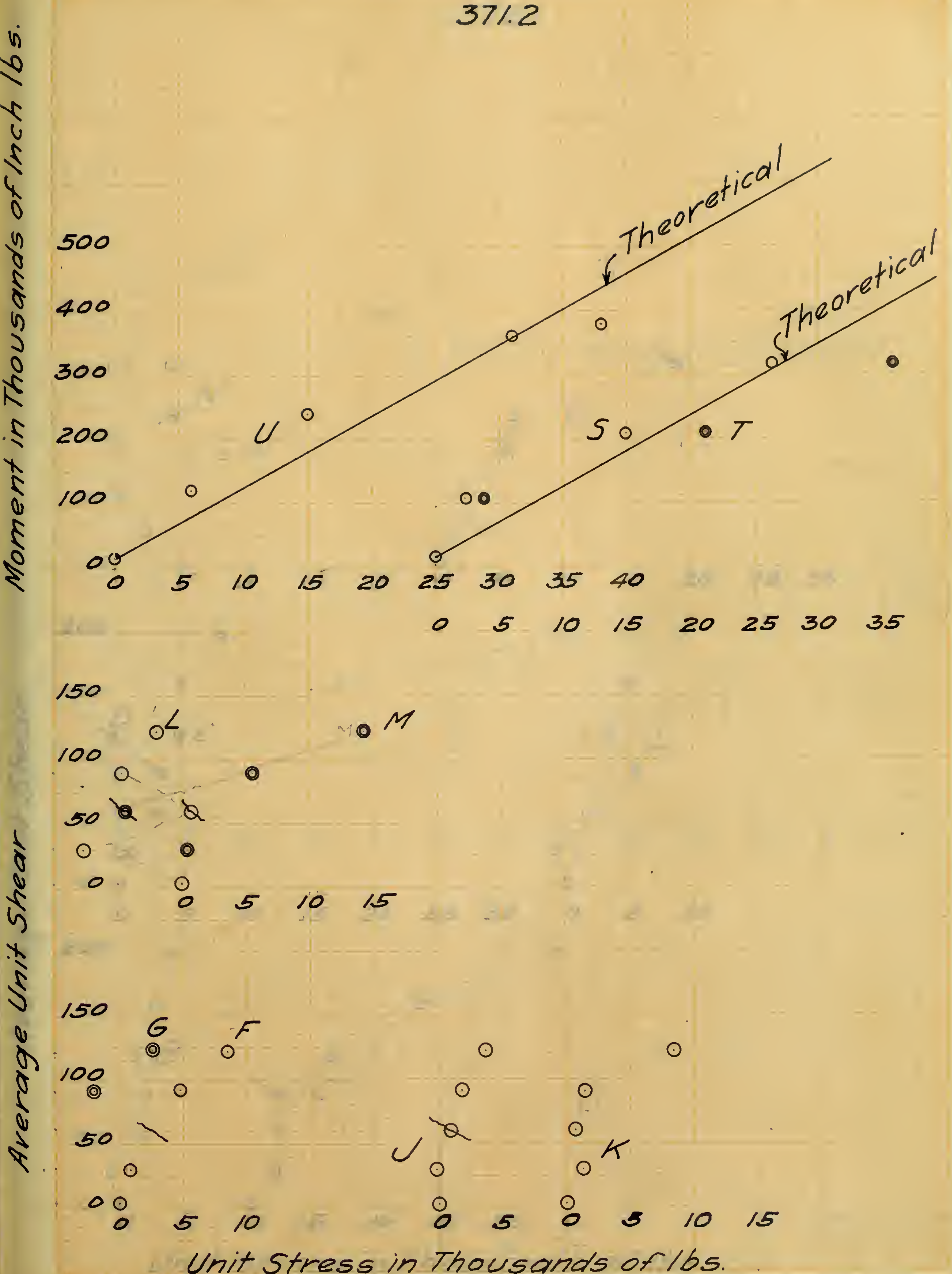
yield point of the steel in some cases. The particular gage lengths showing this high stress can be picked out by keeping in mind the yield point of the steel as given in table I.

High Local Bond Stresses. - It is believed that the local slipping found, particularly of the corrugated bars, shows that there are greater bond stresses developed than are usually considered as being present.

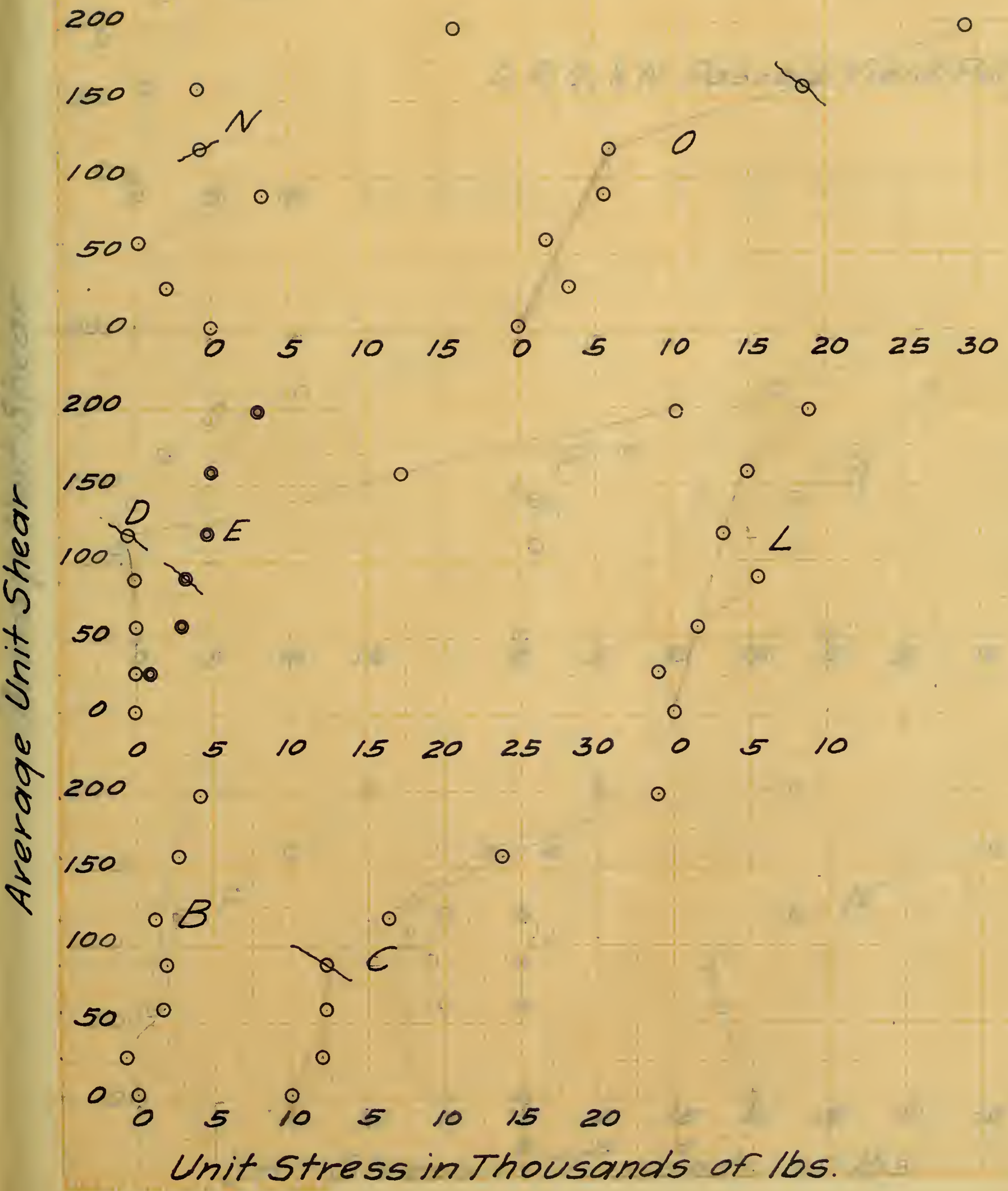
In the limited time available, it will not be possible to go into a discussion of the question of web reinforcement in detail. The foregoing discussion has had reference particularly to third point loading which means uniform shear. The application of the results to the case of beams uniformly loaded would be instructive. Furthermore, the discussion of the tested beam has been confined almost entirely to the action of the portion outside of the support where the so-called diagonal tension failures occurred. The action around the point of inflection, and the beam action in portions of the beam are other points not considered. The proper placing of the web reinforcement can well be studied from the data and discussion given.

It is felt that although exact mathematical calculation of the stresses in web reinforcement may be impossible, yet this series of experiments throws much light upon the action of the reinforced concrete beams in general and suggests lines for further investigation on this subject.

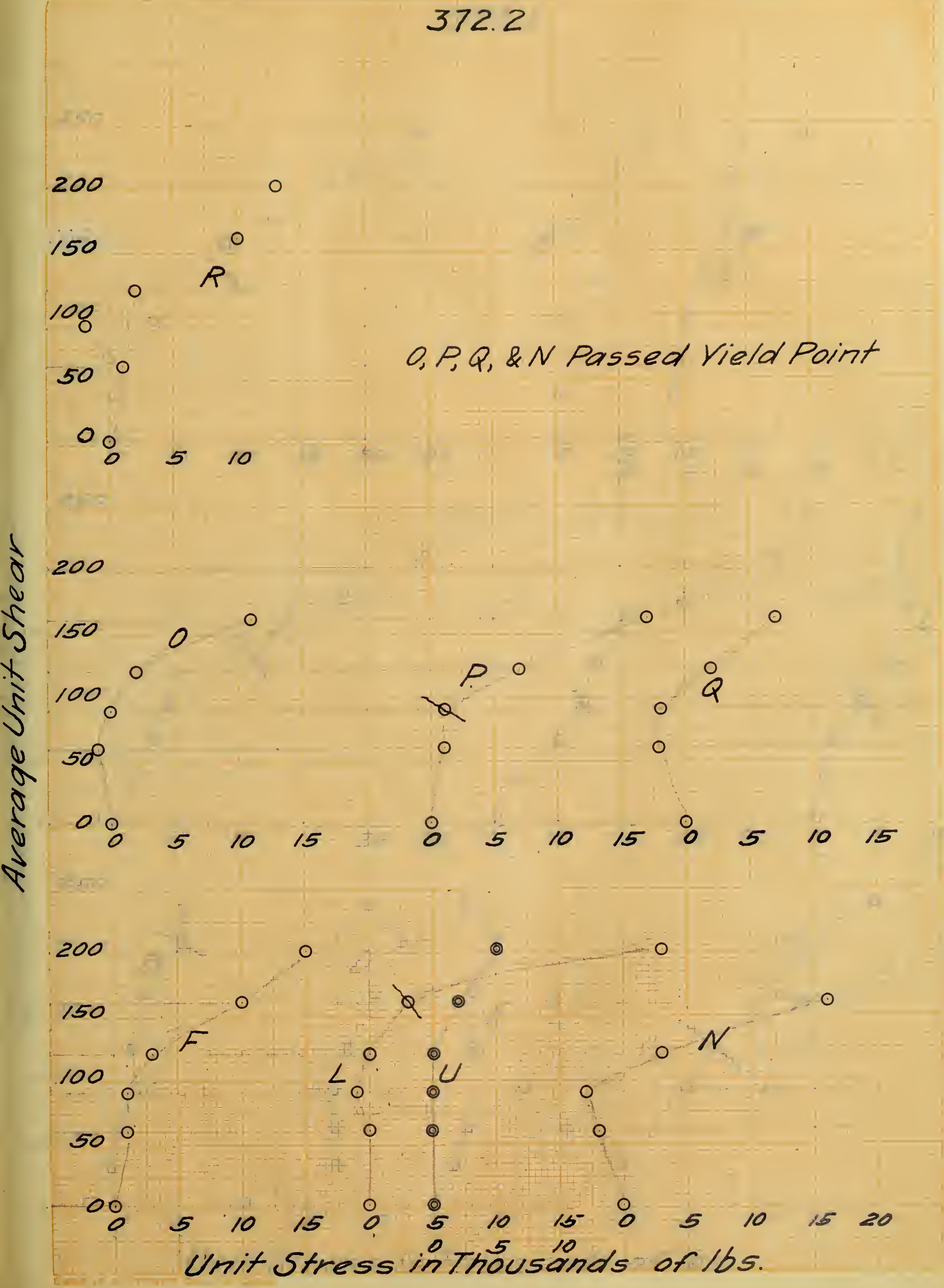
SHEAR – STRESS DIAGRAMS

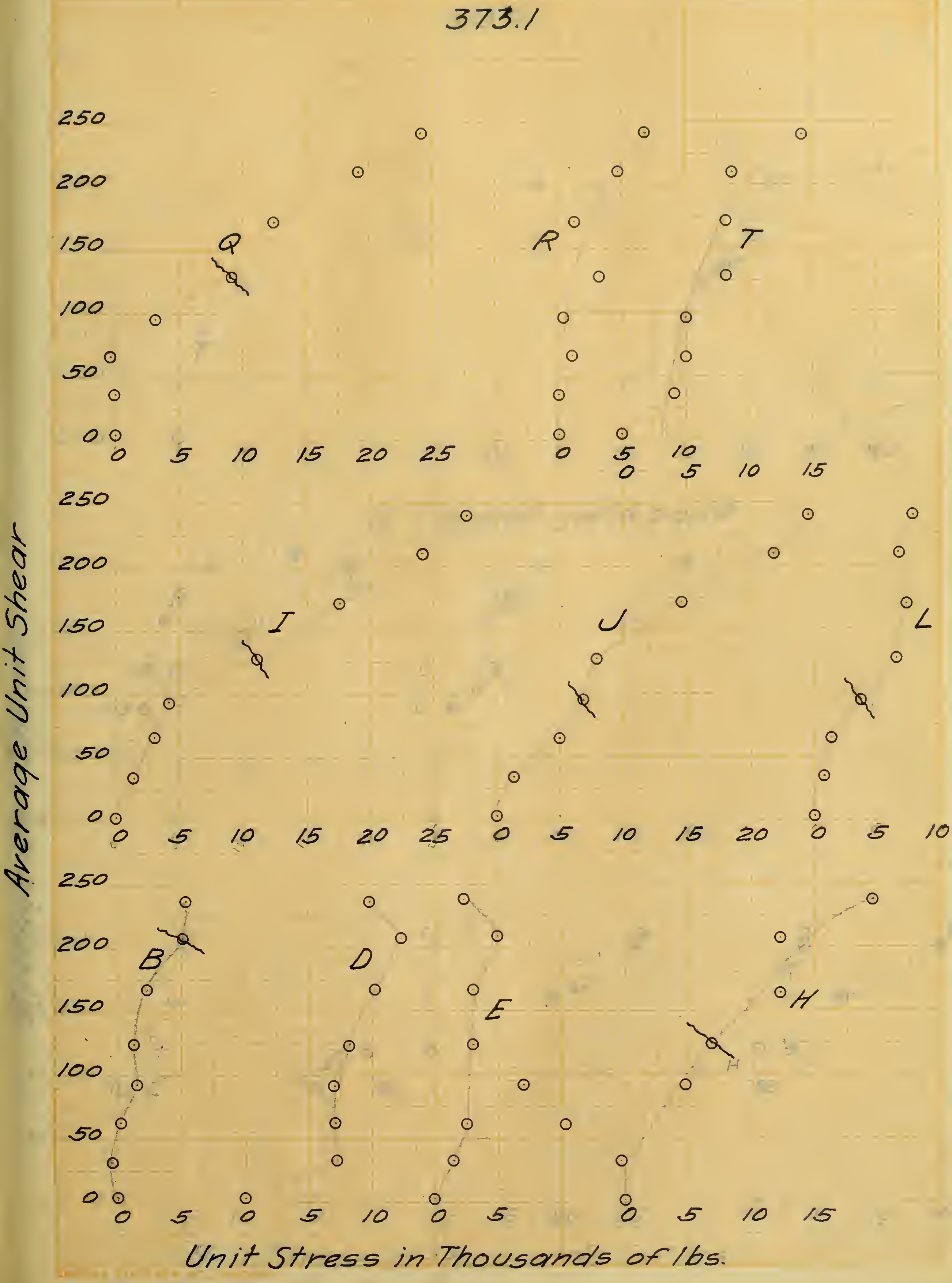

371.2
Moment in Thousands of Inch lbs.
500
400
300
200
100
0
Theoretical
Theoretical
U
S
T
0 5 10 15 20 25 30 35 40
0 5 10 15 20 25 30 35
Average Unit Shear
150
100
50
0
L
M
0 5 10 15
150
100
50
0
G
F
J
K
0 5 10
0 5
0 5 10 15
Unit Stress in Thousands of lbs.

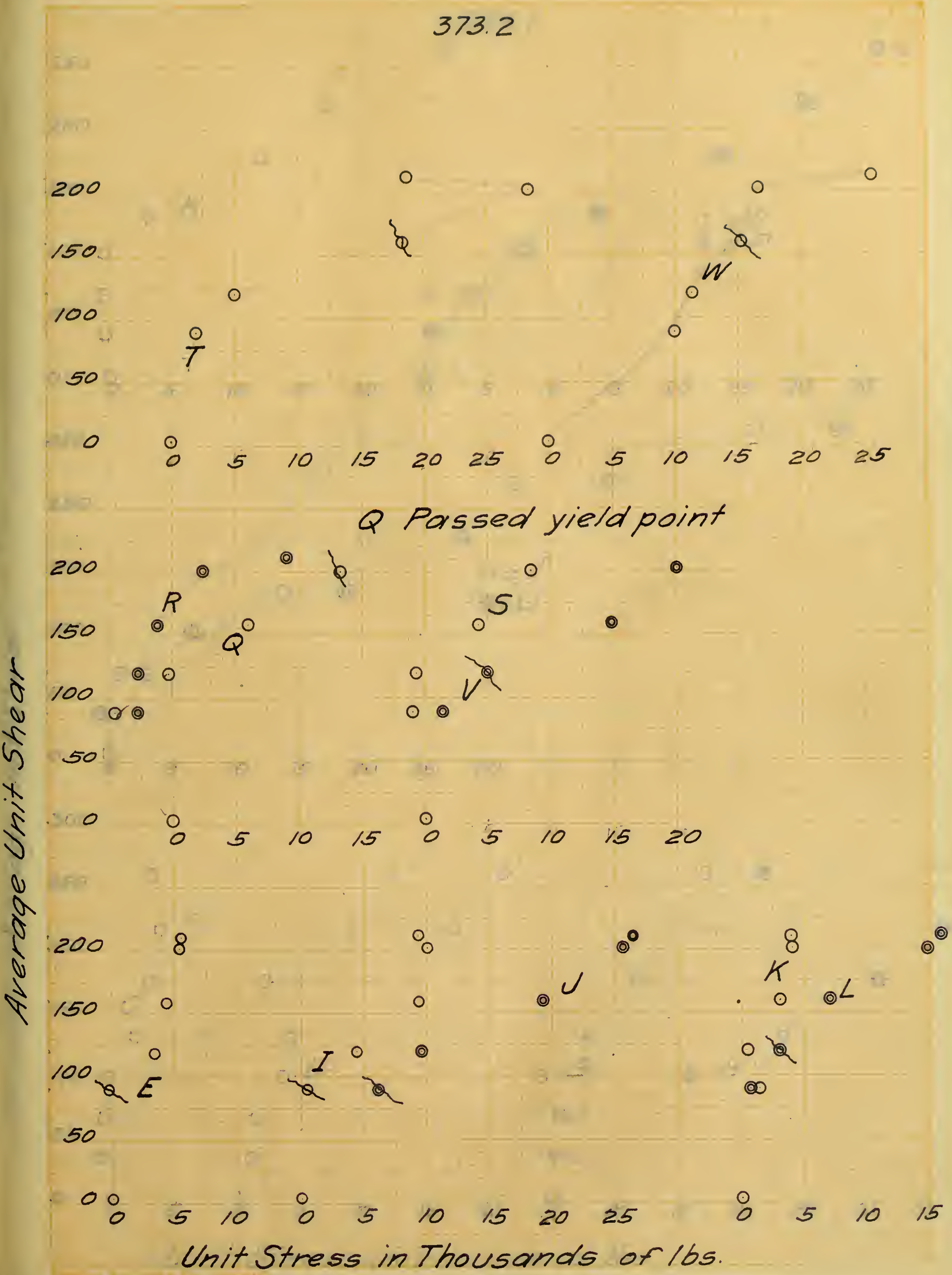
372.1

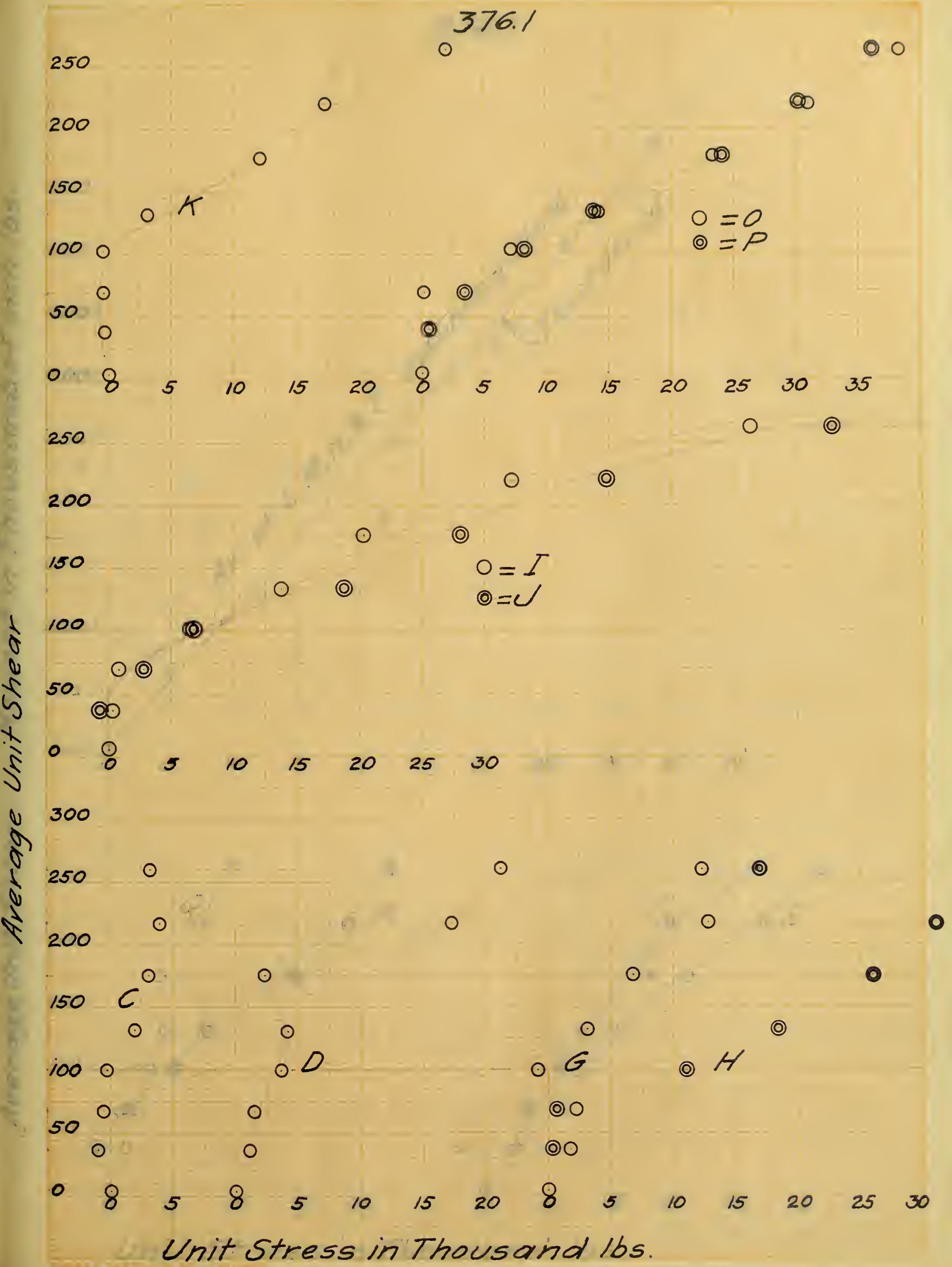


N
O
D
E
L
B
C
200
150
100
50
0
0 5 10 15 0 5 10 15 20 25 30
0 5 10 15 20 25 30 0 5 10
0 5 10 5 10 15 20
Average Unit Shear
Unit Stress in Thousands of lbs.


372.2
O, P, Q, & N Passed Yield Point
Average Unit Shear
200
150
100
50
0
0 5 10
R
O
P
Q
0 5 10 15
F
L
U
N
0 5 10 15 20
Unit Stress in Thousands of lbs.

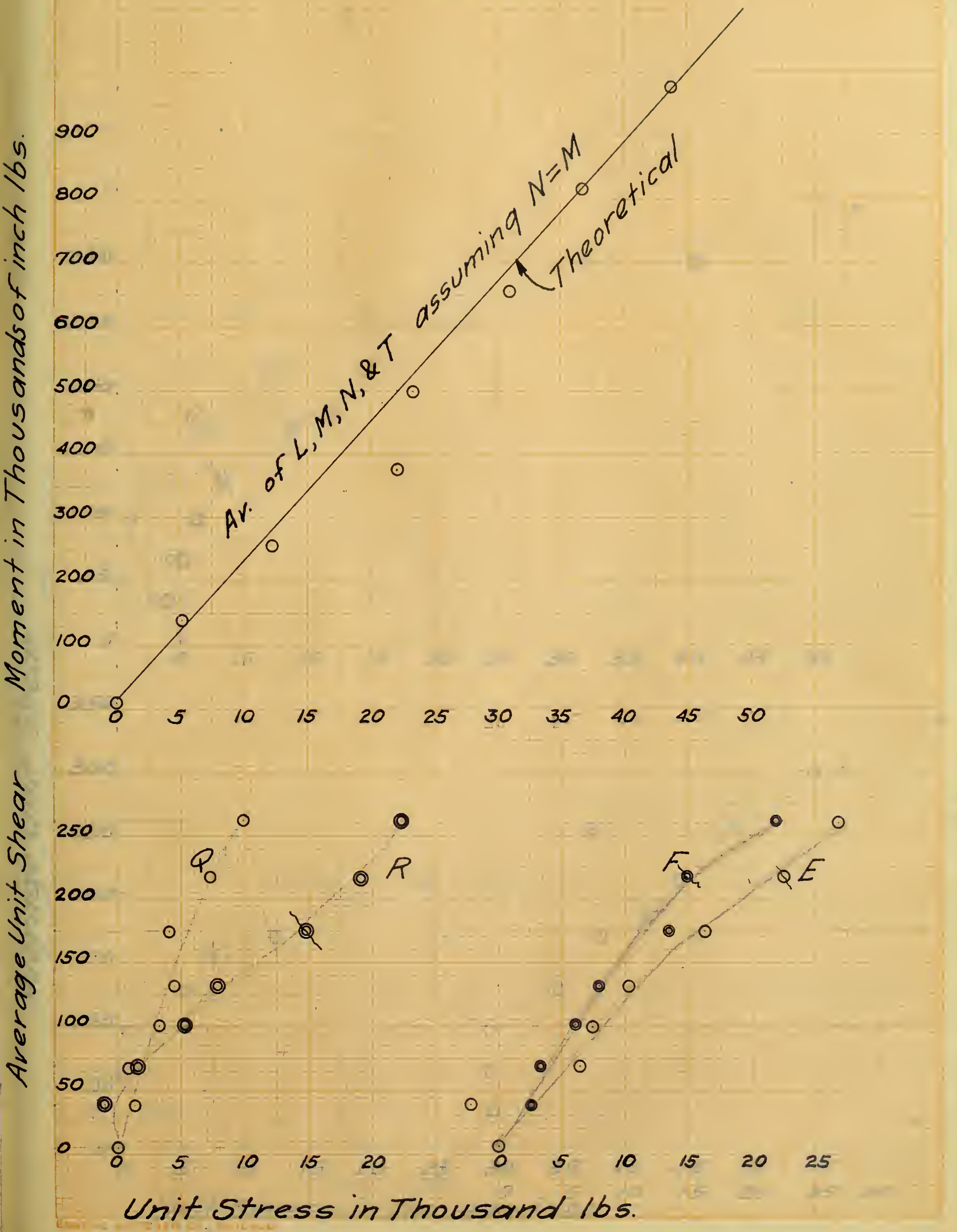

373.1
Q
R
T
I
J
L
B
D
E
H
Average Unit Shear
250
200
150
100
50
0
0 5 10 15 20 25
Unit Stress in Thousands of lbs.


373.2
Average Unit Shear
200
150
100
50
0
T
W
0 5 10 15 20 25 0 5 10 15 20 25
Q Passed yield point
R
Q
S
V
0 5 10 15 0 5 10 15 20
E
I
J
K
L
0 5 10 0 5 10 15 20 25 0 5 10 15
Unit Stress in Thousands of lbs.



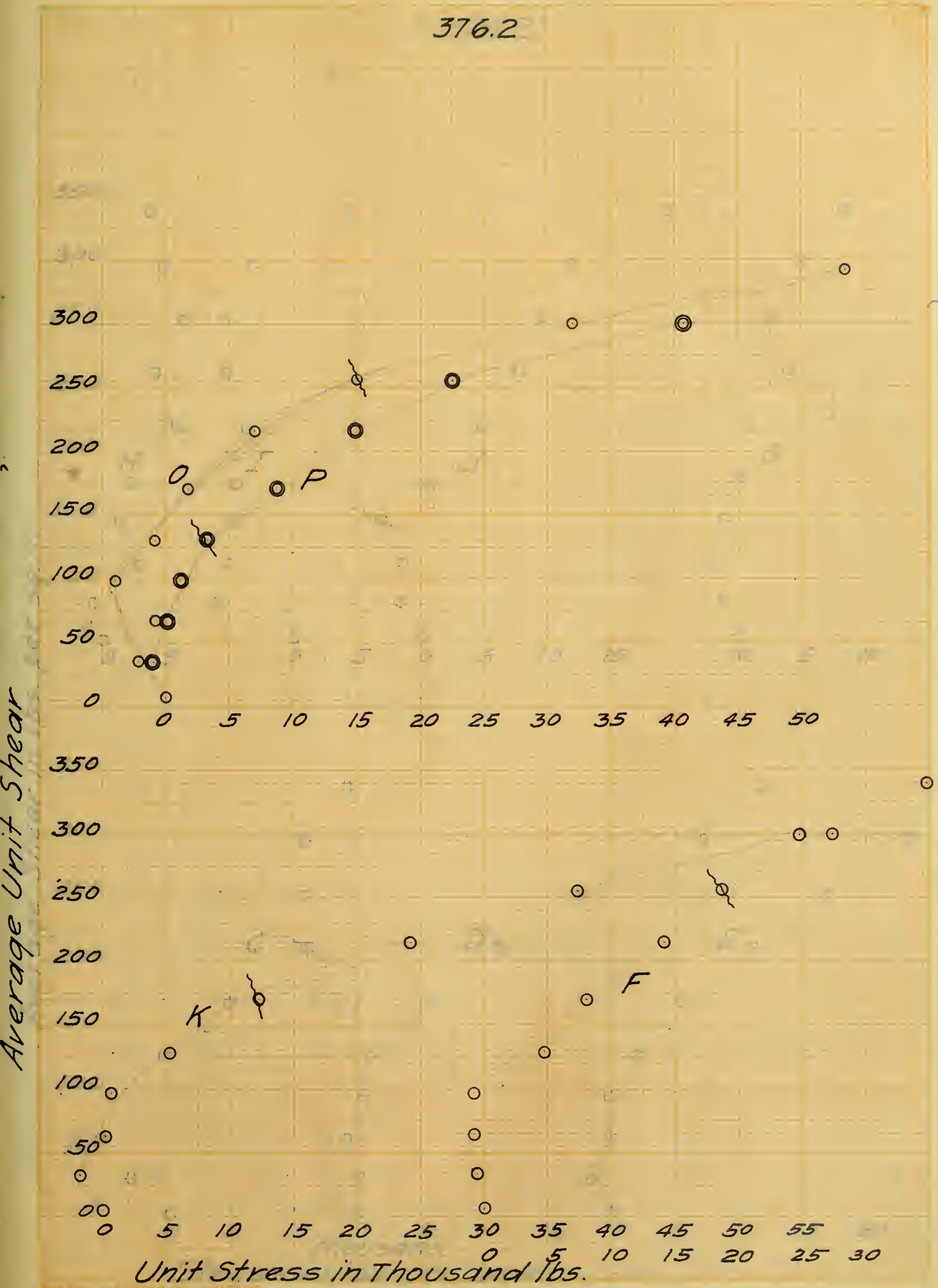
K
○ = O
◎ = P
○ = I
◎ = J
C
D
G
H
Average Unit Shear
Unit Stress in Thousand lbs.

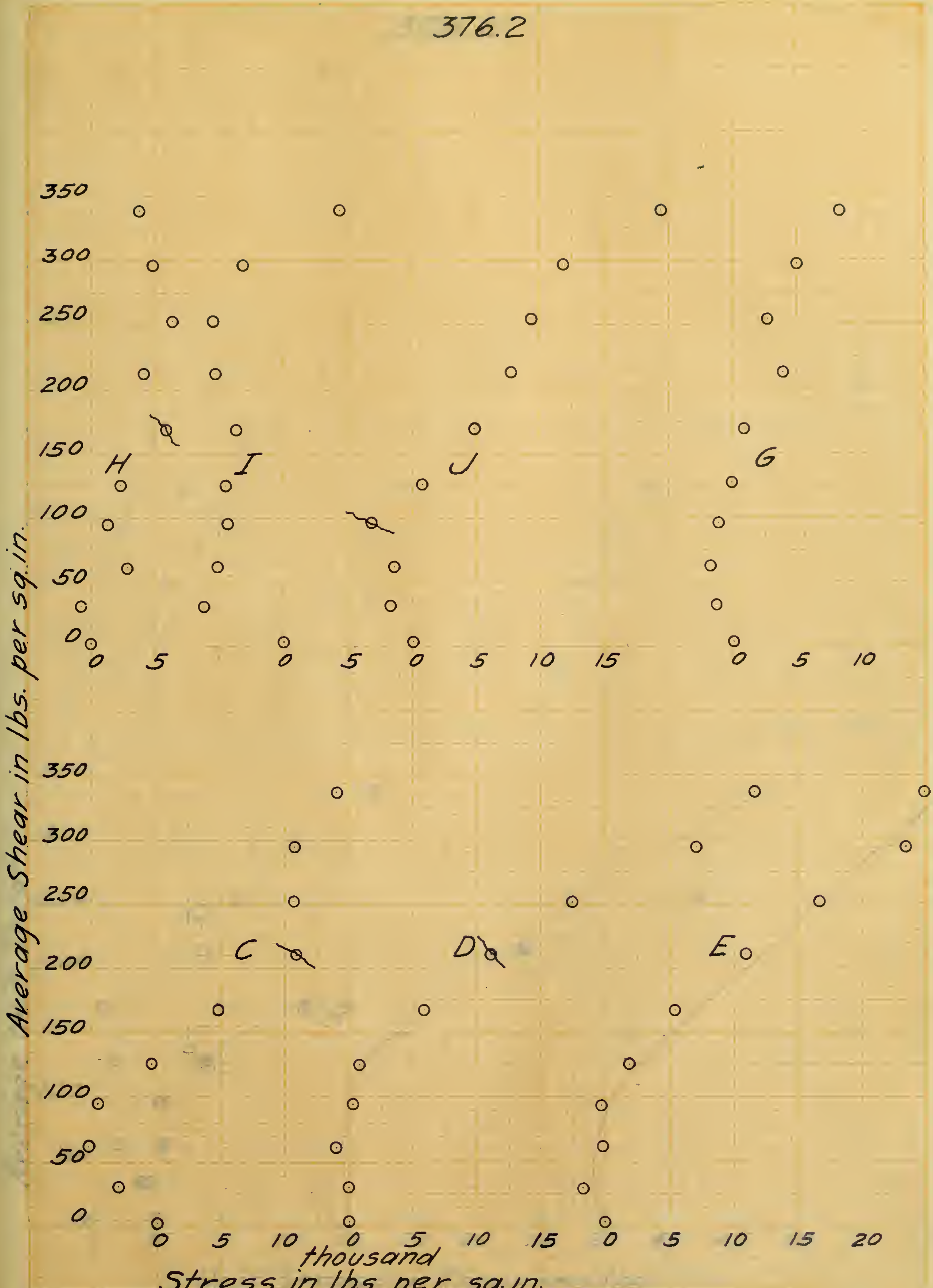
376.1



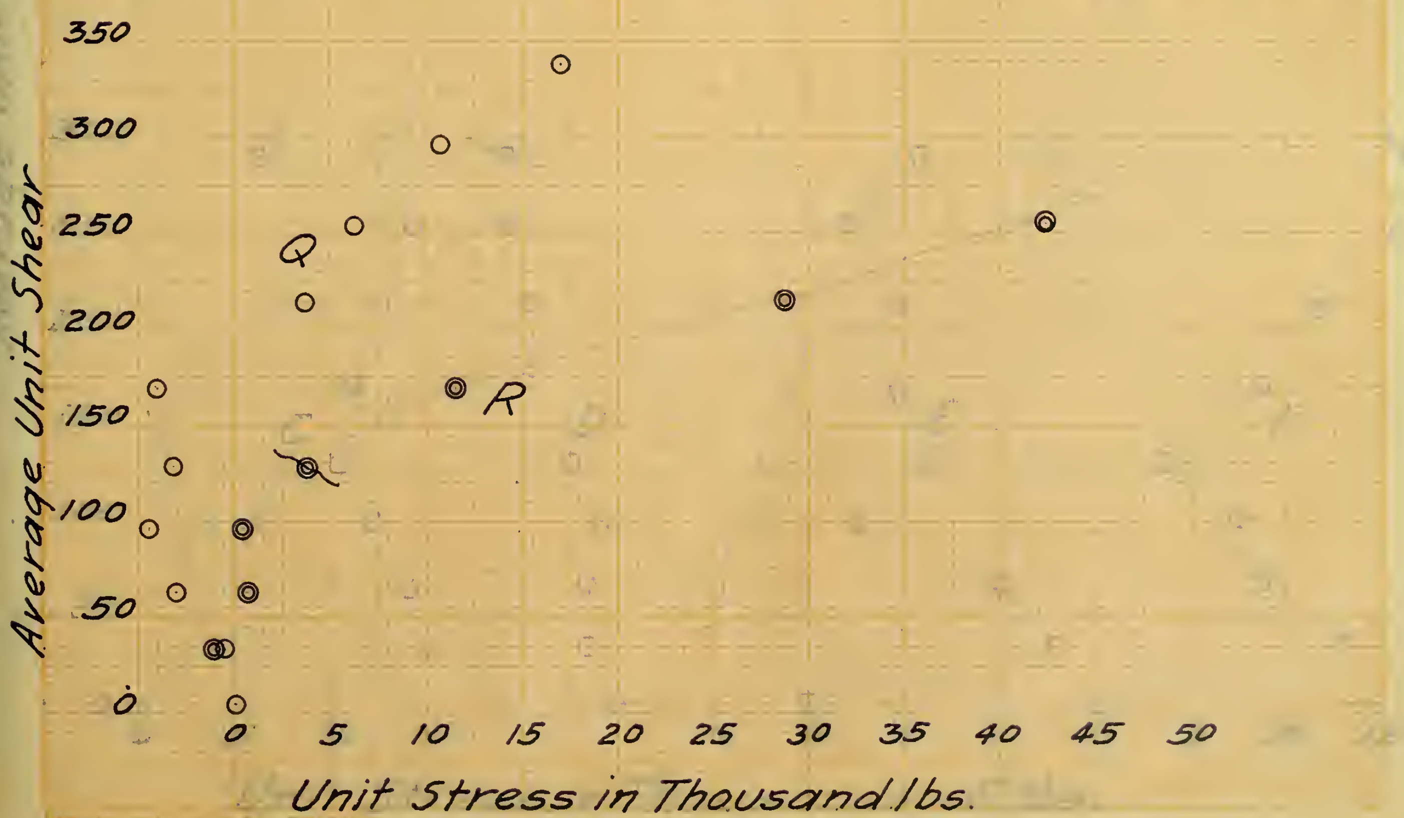
Moment in Thousands of inch lbs.
900
800
700
600
500
400
300
200
100
0
0
5
10
15
20
25
30
35
40
45
50
Av. of L, M, N, & T assuming N=M
Theoretical
Average Unit Shear
250
200
150
100
50
0
0
5
10
15
20
0
5
10
15
20
25
Q
R
F
E
Unit Stress in Thousand lbs.

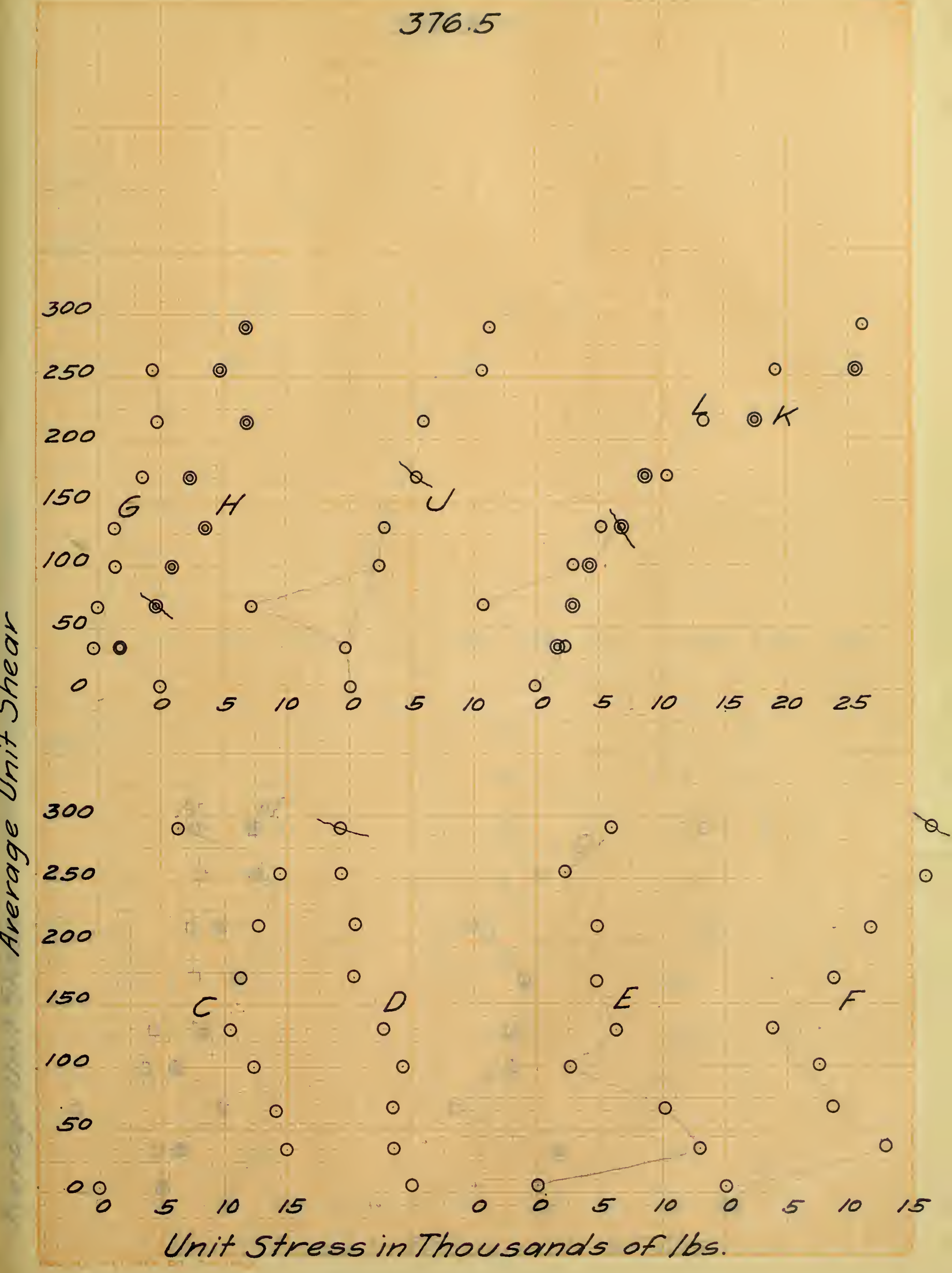
376.2

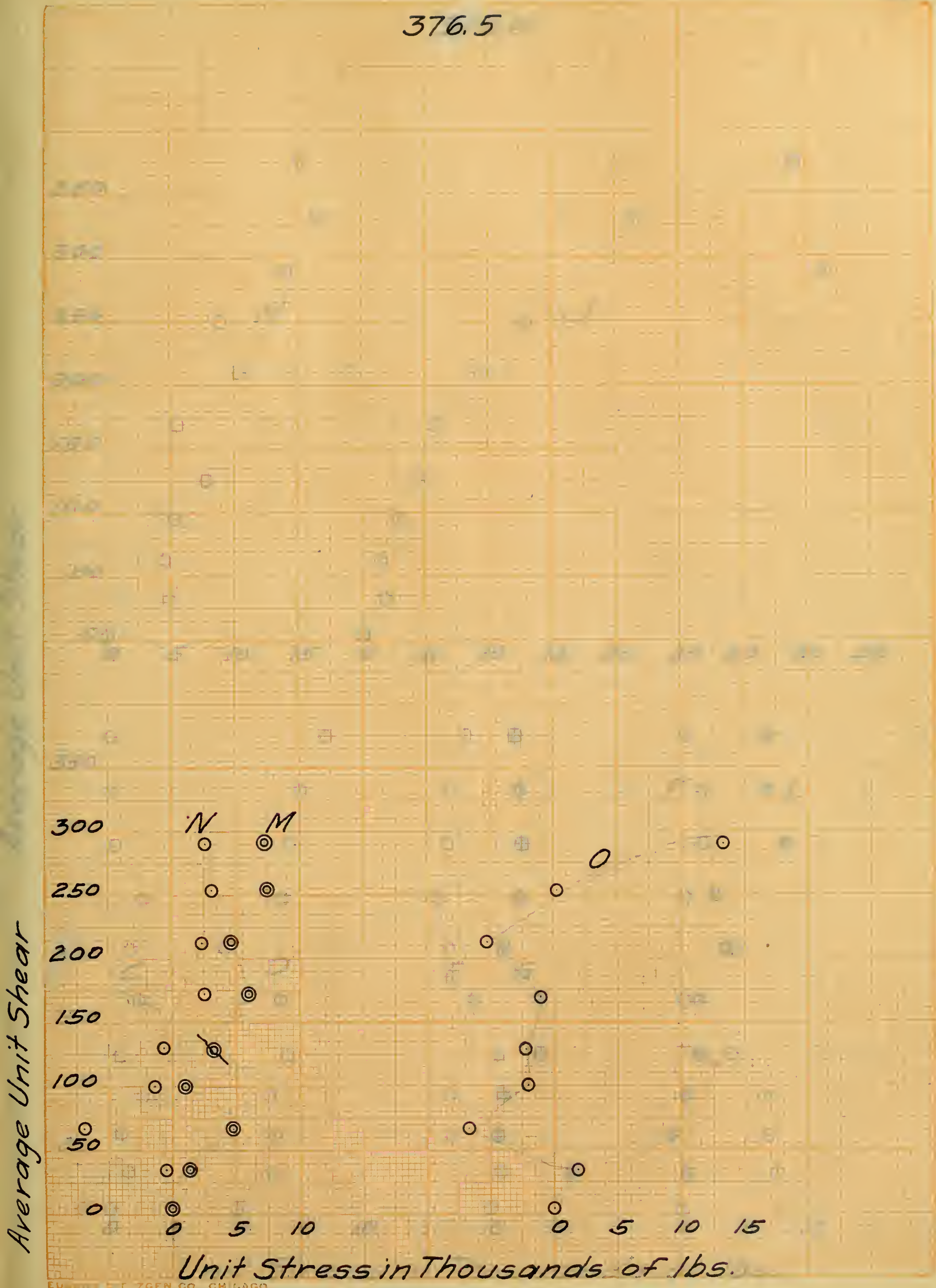

O
P
K
F
300
250
200
150
100
50
0
0 5 10 15 20 25 30 35 40 45 50
350
300
250
200
150
100
50
0
0 5 10 15 20 25 30 35 40 45 50 55
0 5 10 15 20 25 30
Average Unit Shear
Unit Stress in Thousand lbs.

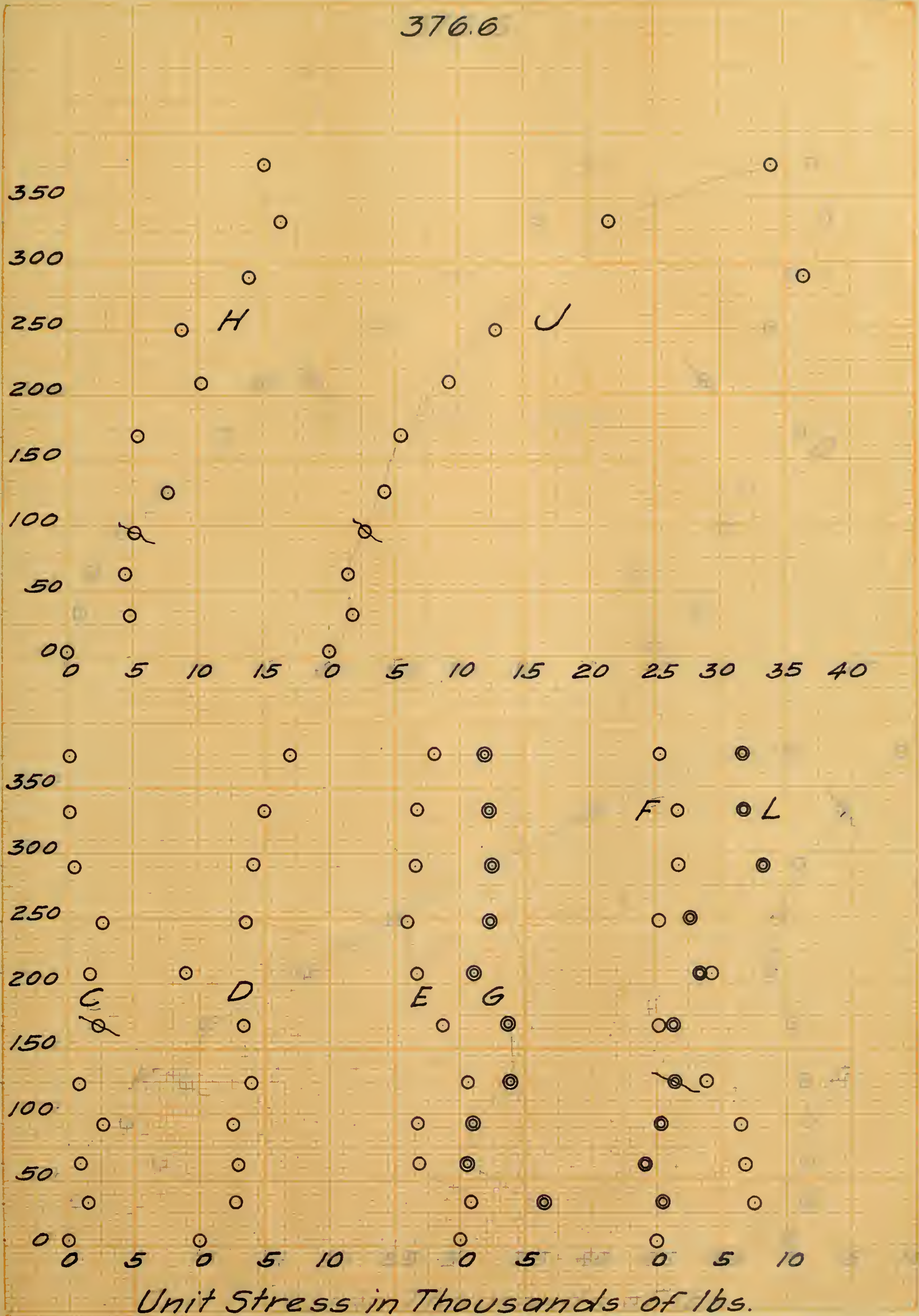

376.2
H
I
J
G
C
D
E
350
300
250
200
150
100
50
0
0 5 0 5 0 5 10 15 0 5 10
0 5 10 0 5 10 15 0 5 10 15 20
Average Shear in lbs. per sq. in.
Stress in thousand lbs. per sq. in.

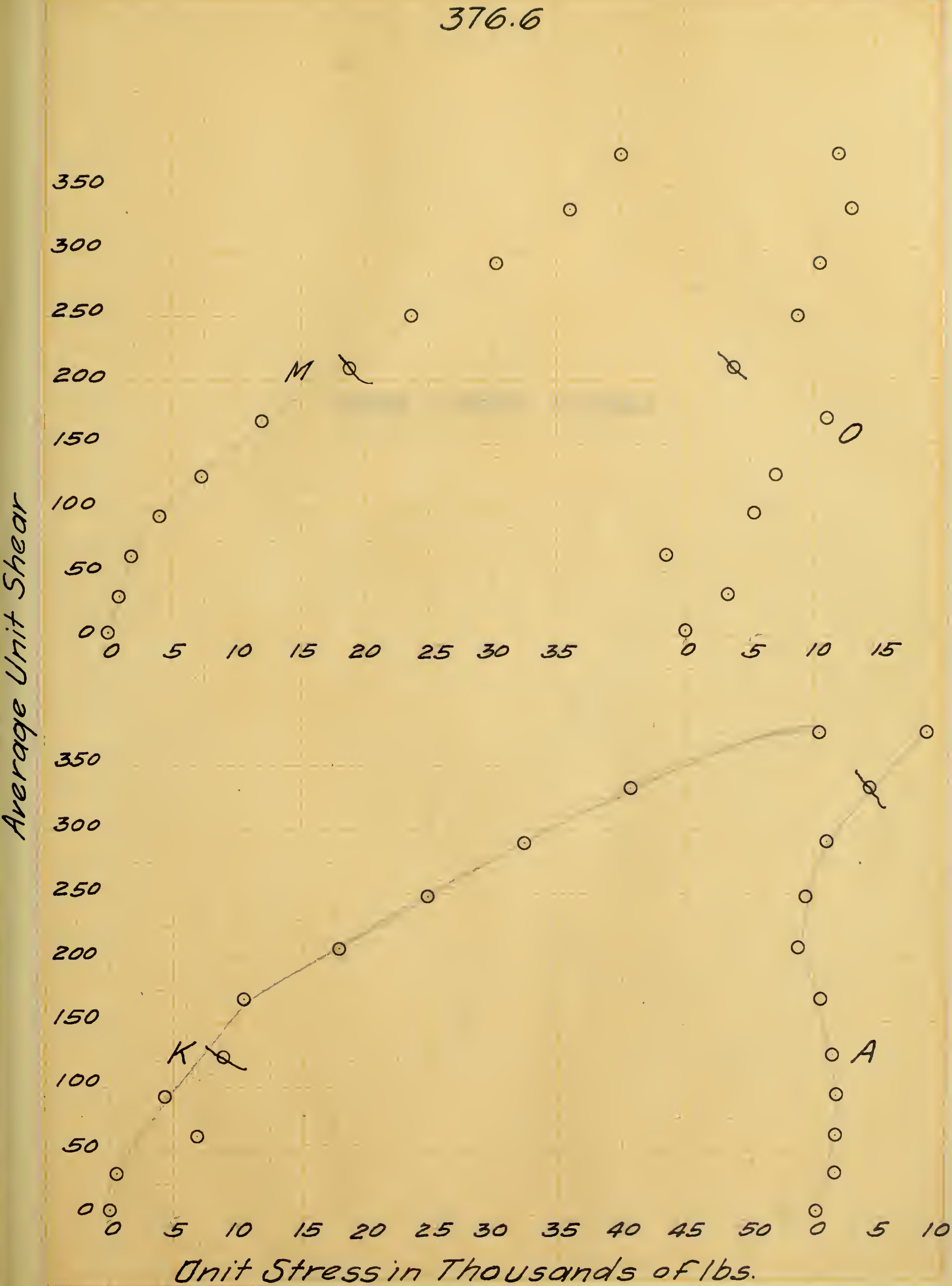
376.2


350
300
250
200
150
100
50
0
Q
R
Average Unit Shear
0
5
10
15
20
25
30
35
40
45
50
Unit Stress in Thousand lbs.


376.5
Average Unit Shear
300
250
200
150
100
50
0
G
H
J
K
0
5
10
15
20
25
C
D
E
F
Unit Stress in Thousands of lbs.

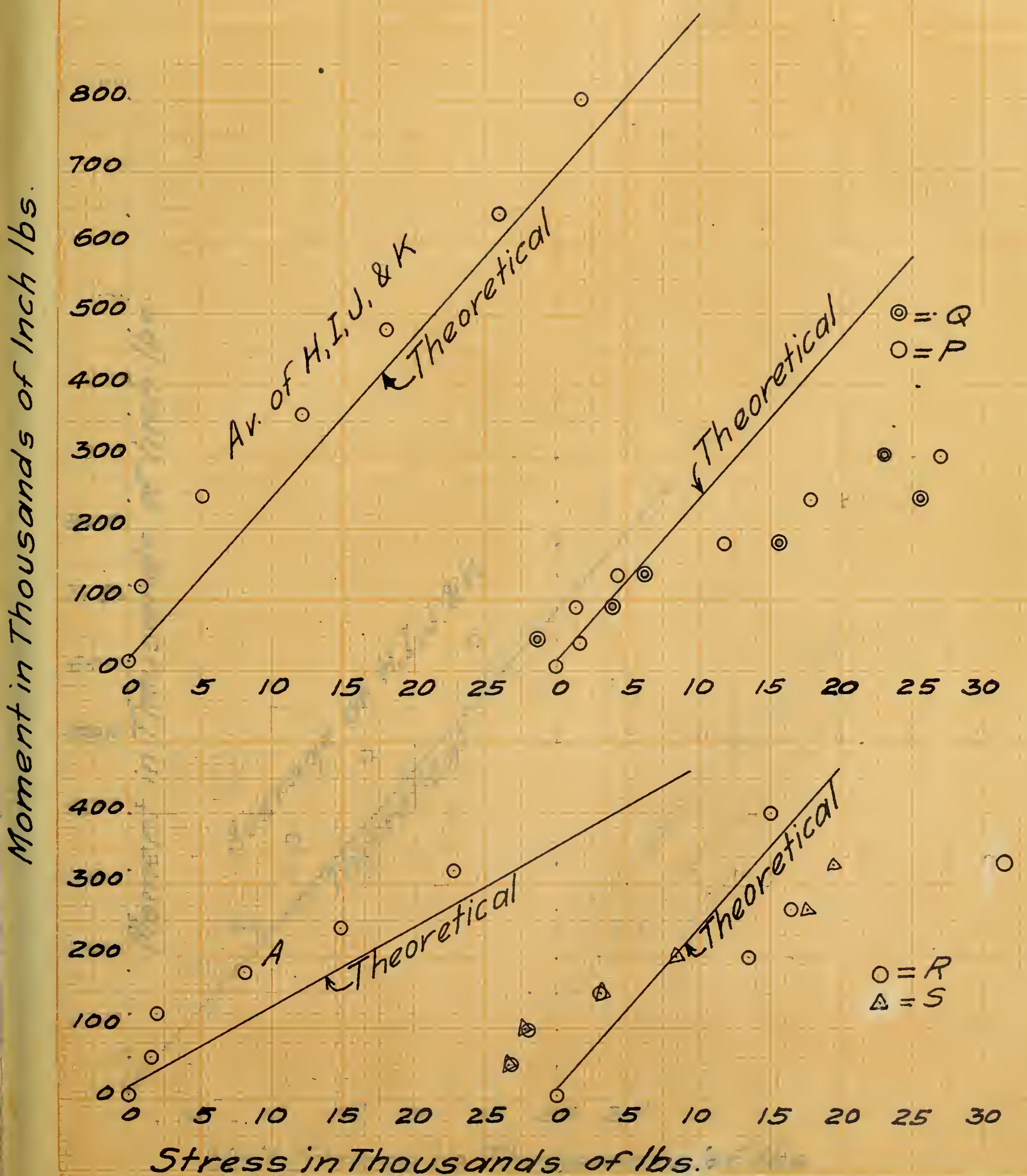

376.5
N
M
O
300
250
200
150
100
50
0
0
5
10
0
5
10
15
Average Unit Shear
Unit Stress in Thousands of lbs.


376.6
H
J
C
D
E
G
F
L
Average Unit Shear
350
300
250
200
150
100
50
0
0 5 10 15 0 5 10 15 20 25 30 35 40
0 5 0 5 10 0 5 0 5 10
Unit Stress in Thousands of lbs.

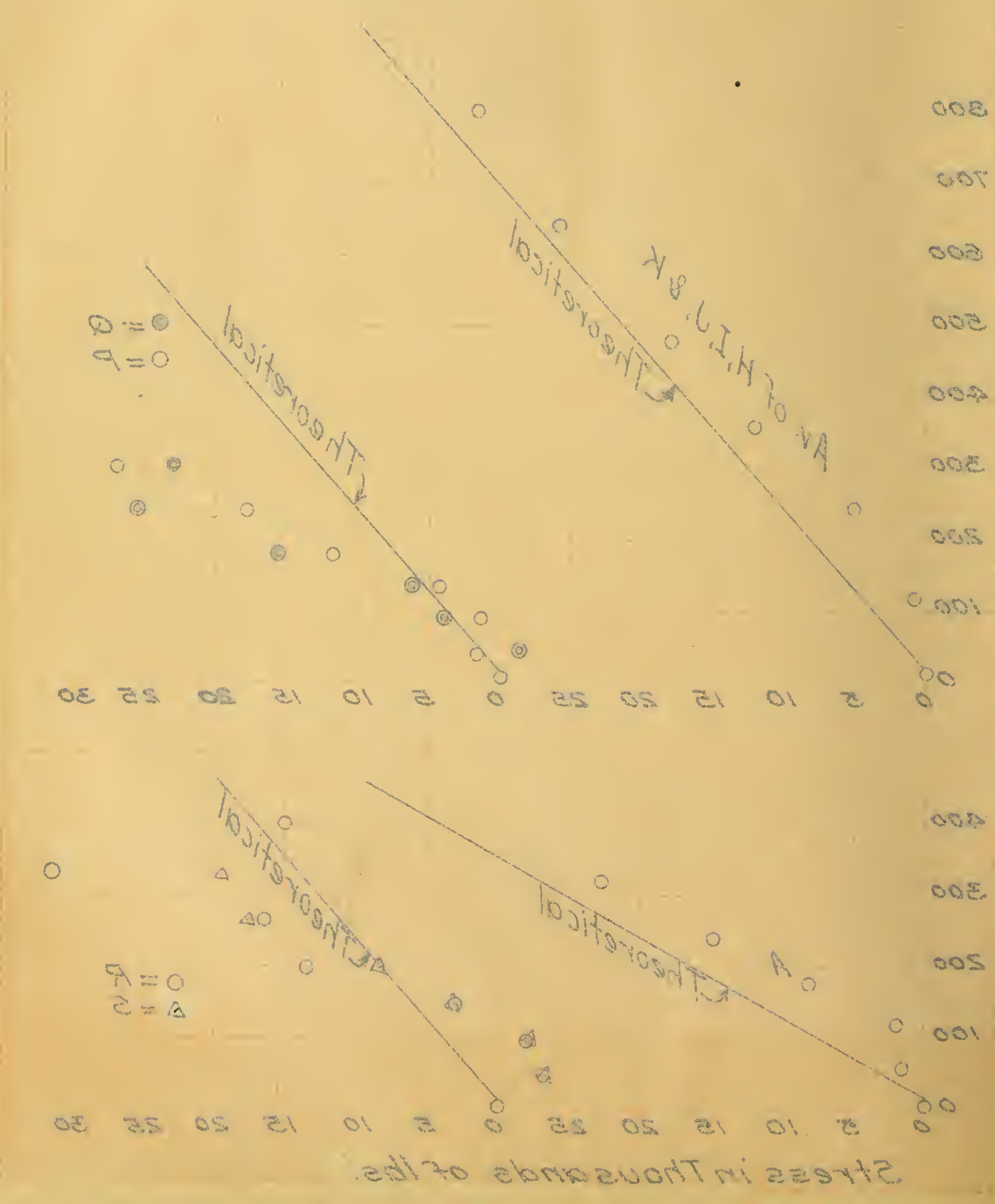

376.6
Average Unit Shear
350
300
250
200
150
100
50
0
M
O
0
5
10
15
20
25
30
35
0
5
10
15
K
A
40
45
50
Unit Stress in Thousands of lbs.

MOMENT - STRESS DIAGRAMS

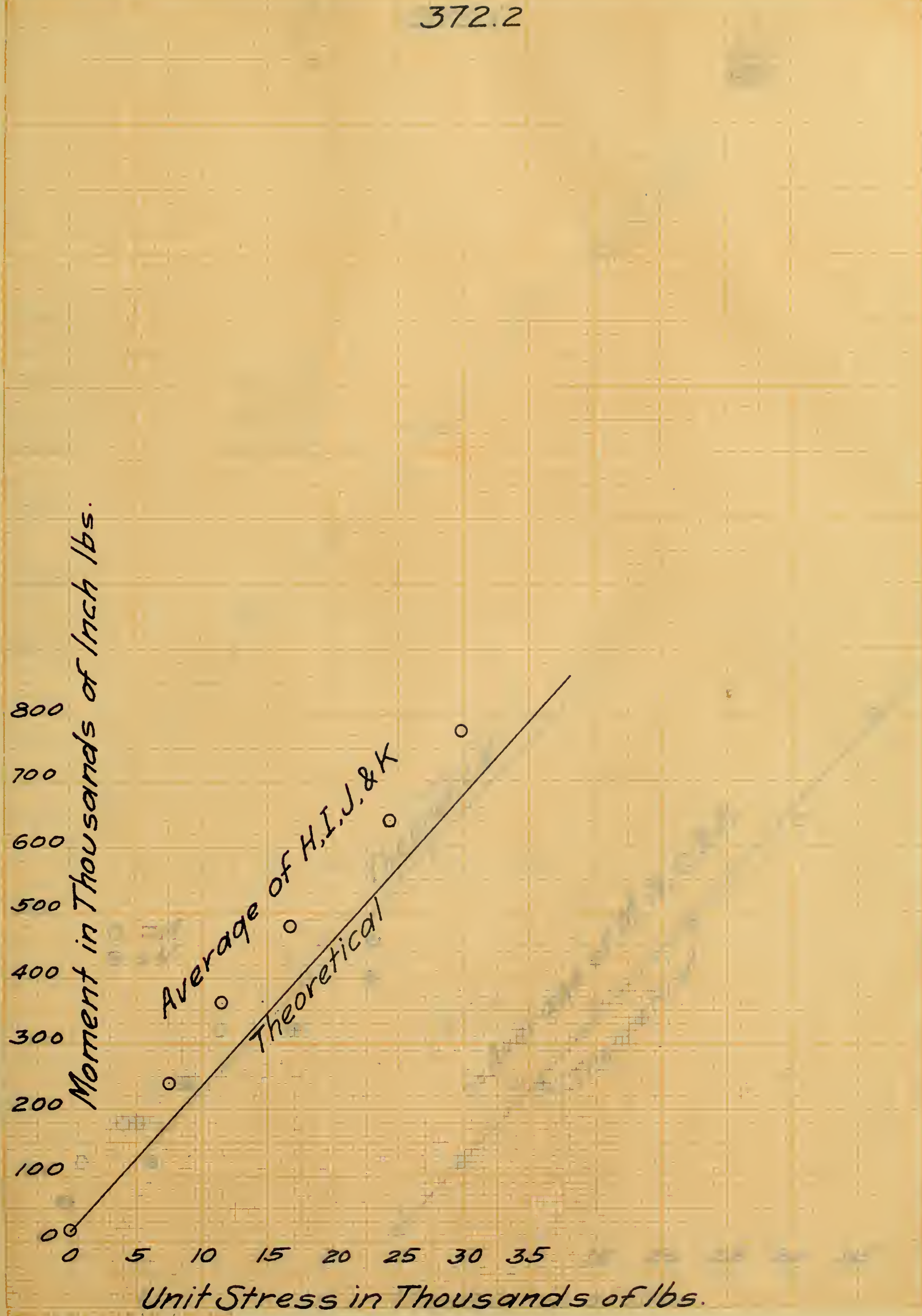
372.1


Moment in Thousands of Inch lbs.
800
700
600
500
400
300
200
100
0
0 5 10 15 20 25 0 5 10 15 20 25 30
Av. of H, I, J, & K
Theoretical
Theoretical
◎ = Q
○ = P
400
300
200
100
0
0 5 10 15 20 25 0 5 10 15 20 25 30
A
Theoretical
Theoretical
○ = R
△ = S
Stress in Thousands of lbs.

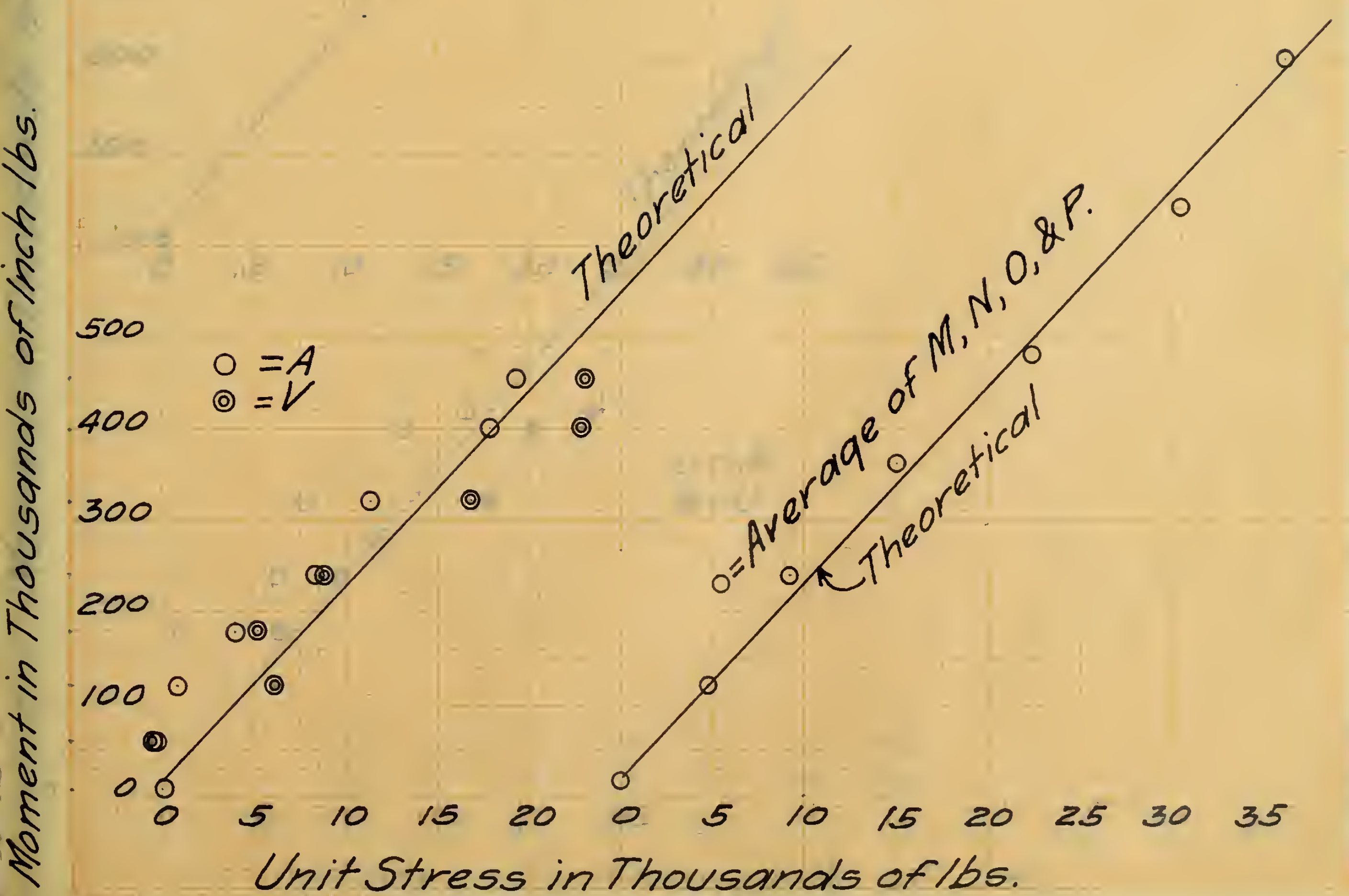
372.1



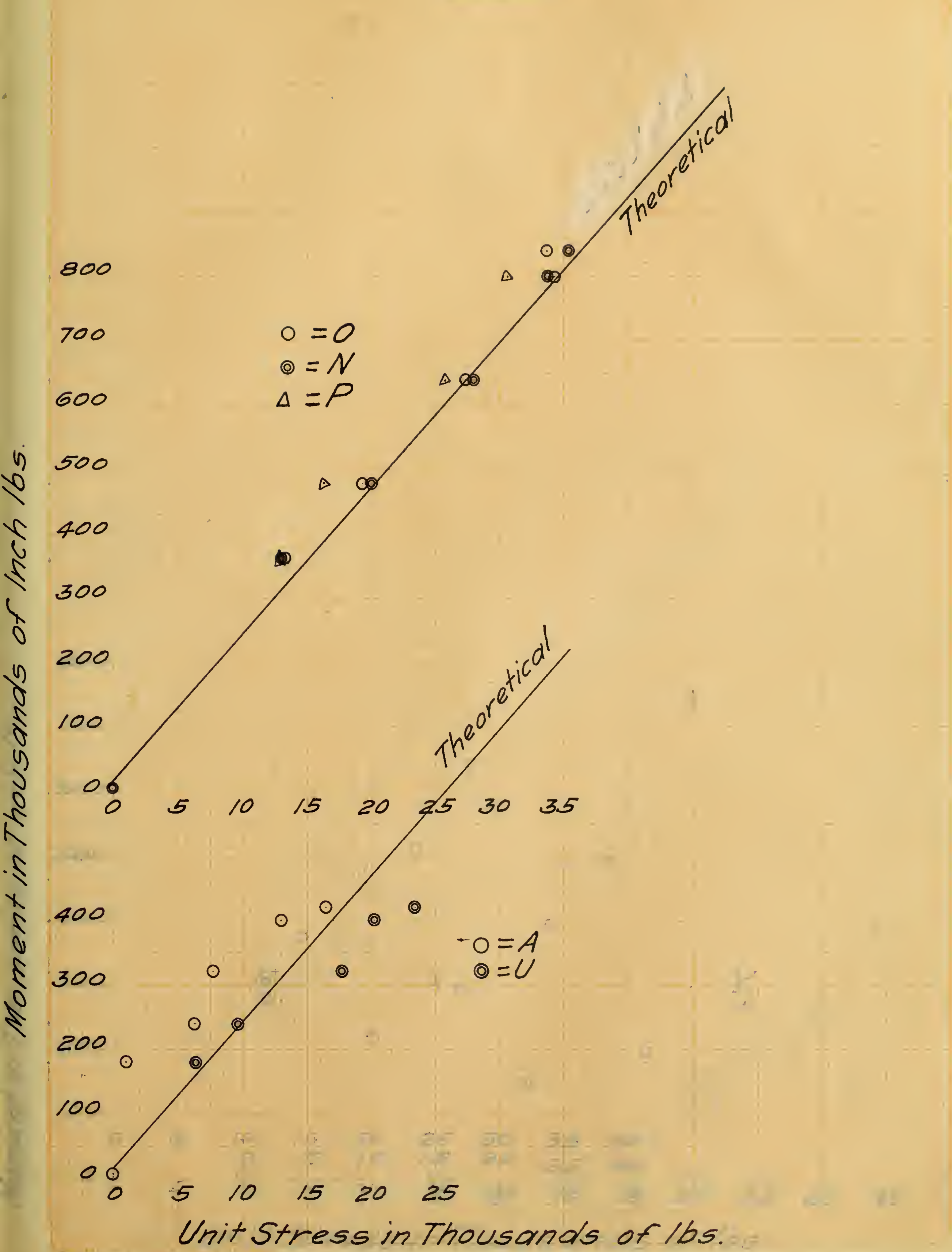
Av. of H.I.J. & K
Theoretical
Theoretical
Theoretical
Theoretical
Q
P
A
S
Stress in Thousands of lbs.

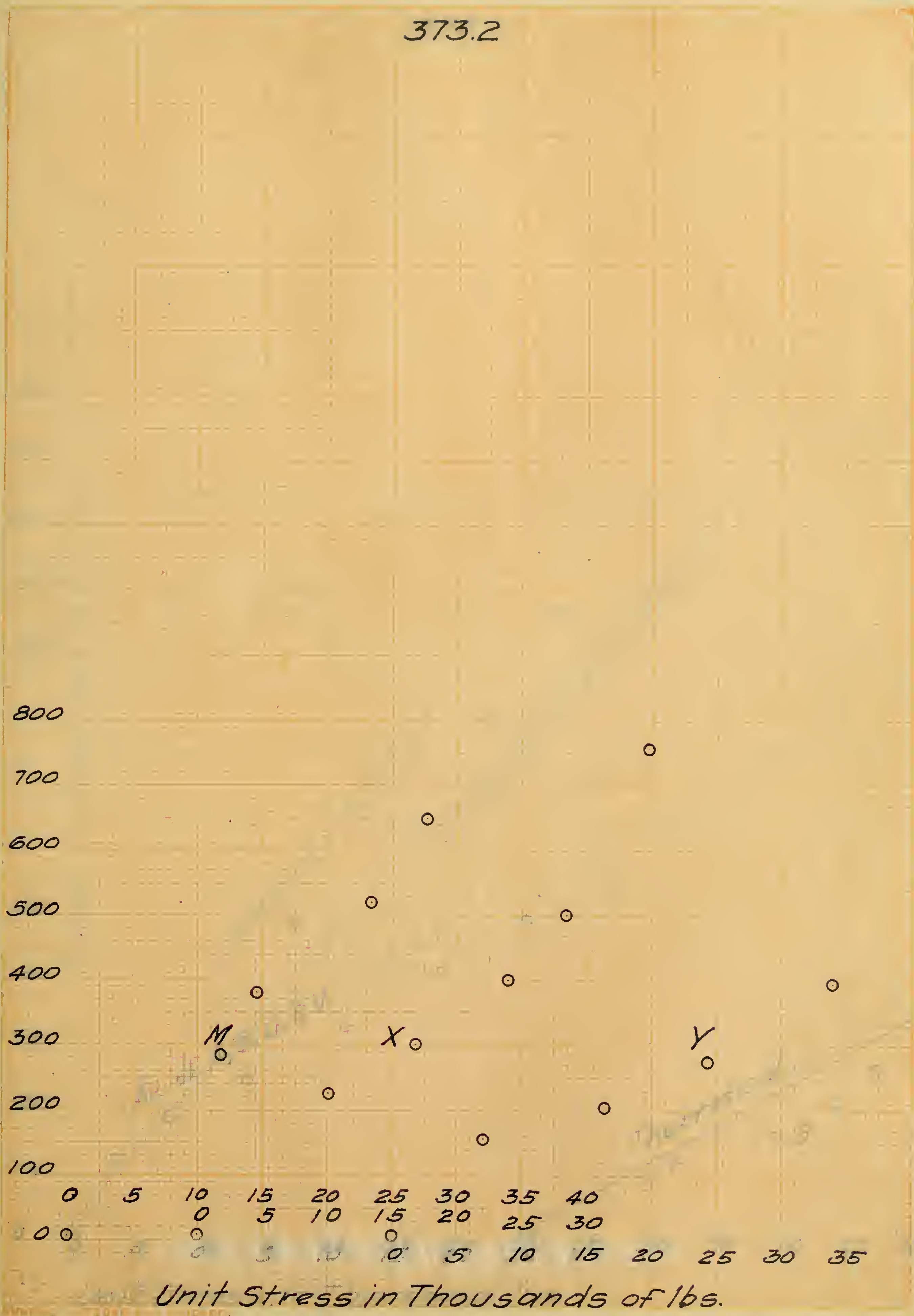

372.2
Moment in Thousands of Inch lbs.
800
700
600
500
400
300
200
100
0
Average of H, I, J, & K
Theoretical
0
5
10
15
20
25
30
35
Unit Stress in Thousands of lbs.

373.1

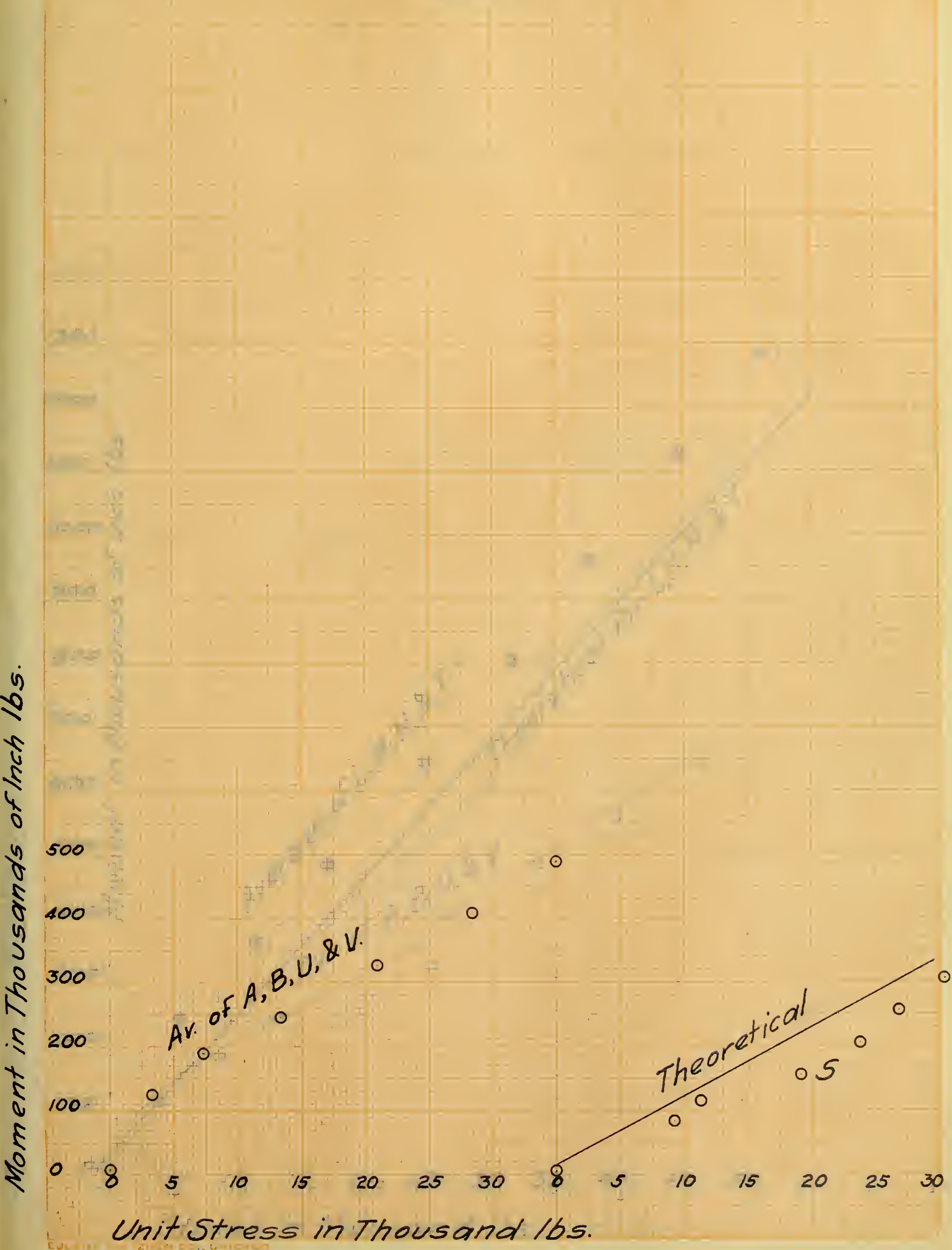

Theoretical
Theoretical
○ =A
◎ =V
○=Average of M, N, O, & P.
500
400
300
200
100
0
0 5 10 15 20
0 5 10 15 20 25 30 35
Moment in Thousands of inch lbs.
Unit Stress in Thousands of lbs.

373.2

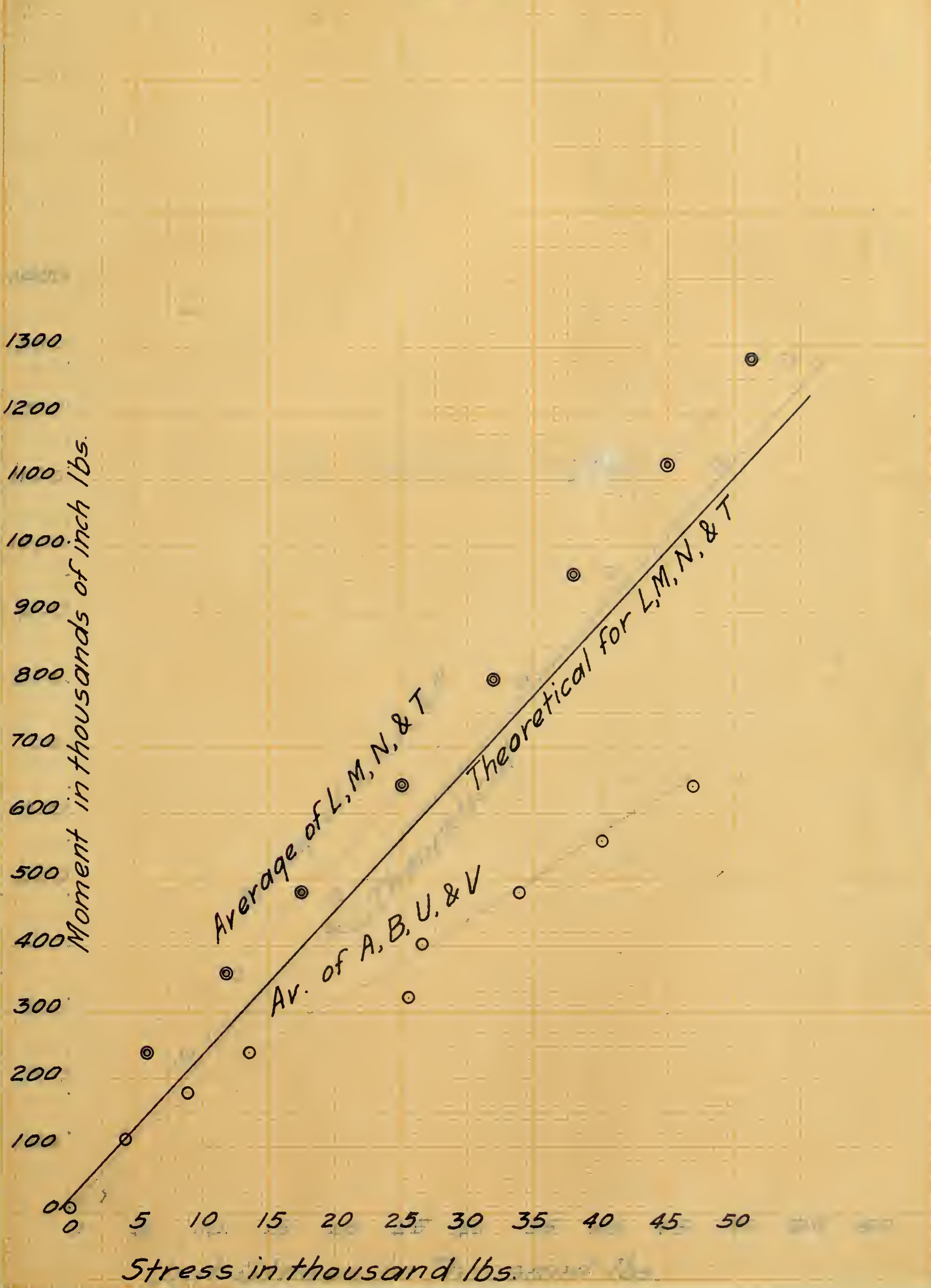

Theoretical
Theoretical
800
700
600
500
400
300
200
100
0
○ = O
◎ = N
Δ = P
0 5 10 15 20 25 30 35
400
300
200
100
0
○ = A
◎ = U
0 5 10 15 20 25
Moment in Thousands of Inch lbs.
Unit Stress in Thousands of lbs.


373.2
Moment in Thousands of Inch lbs.
800
700
600
500
400
300
200
100
0
M
X
Y
0 5 10 15 20 25 30 35 40
0 5 10 15 20 25 30
0 5 10 15 20 25 30 35
Unit Stress in Thousands of lbs.

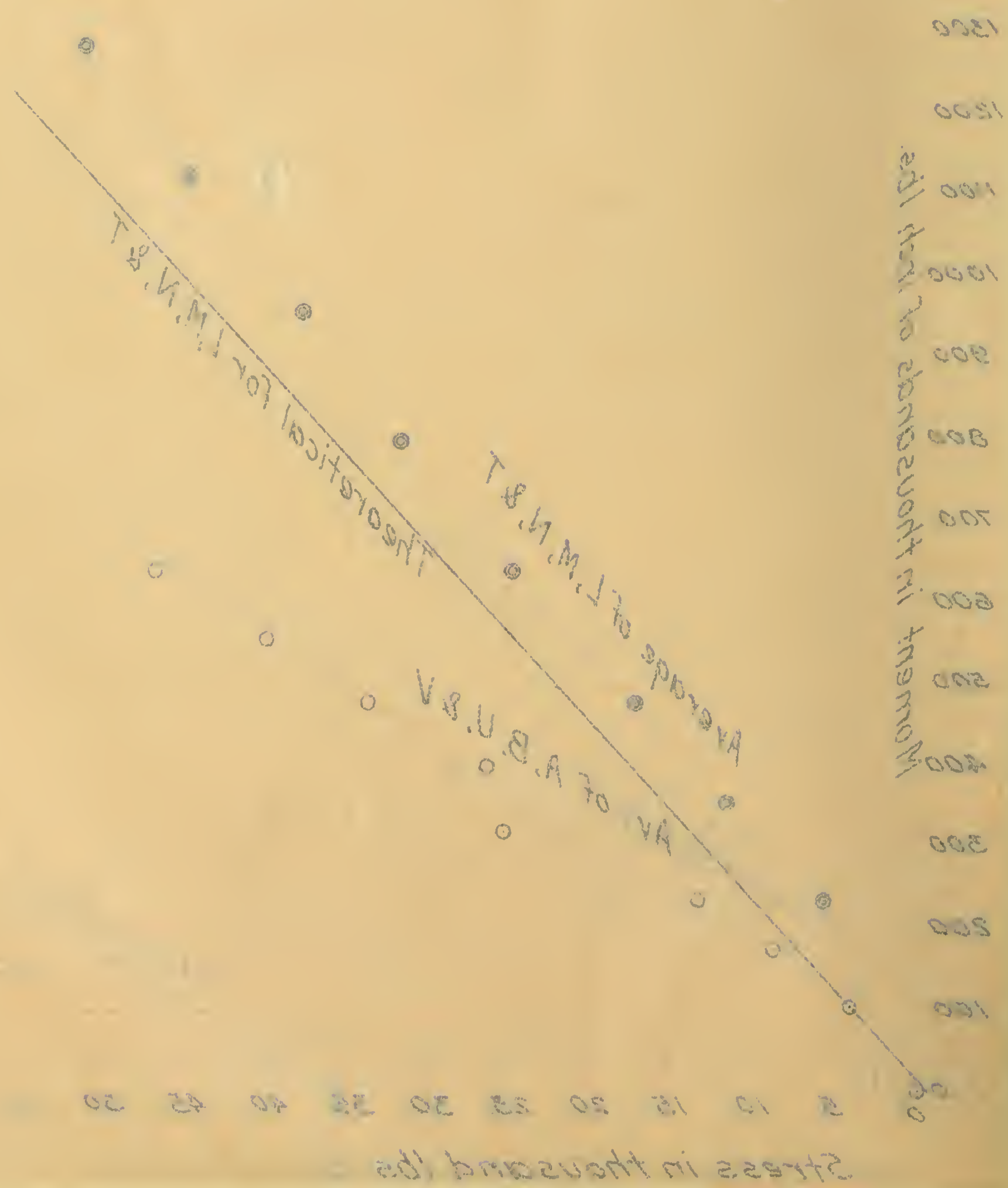
376.1

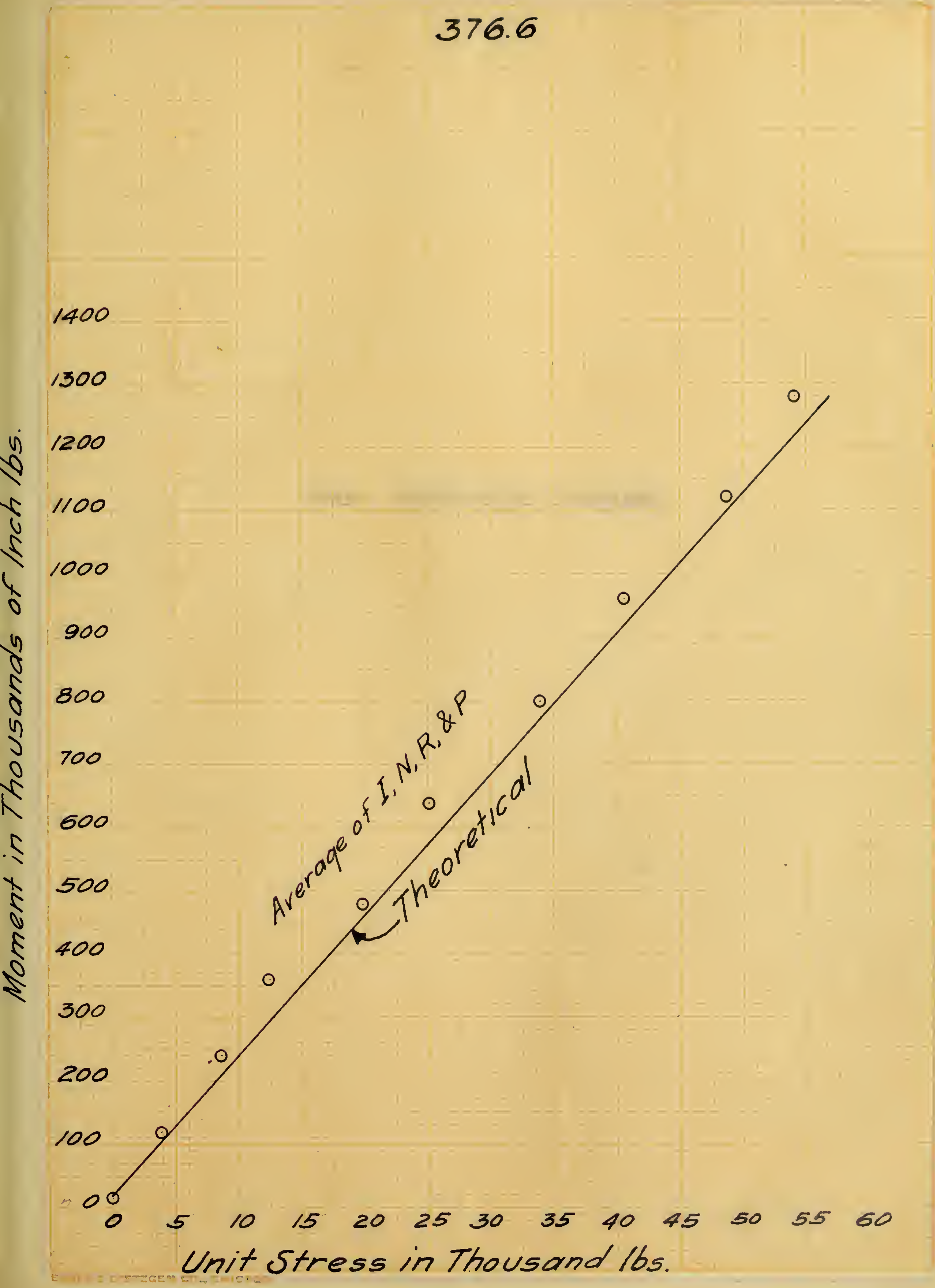

Moment in Thousands of Inch lbs.
500
400
300
200
100
0
Av. of A, B, U, & V.
Theoretical
S
0 5 10 15 20 25 30
0 5 10 15 20 25 30
Unit Stress in Thousand lbs.

376.2

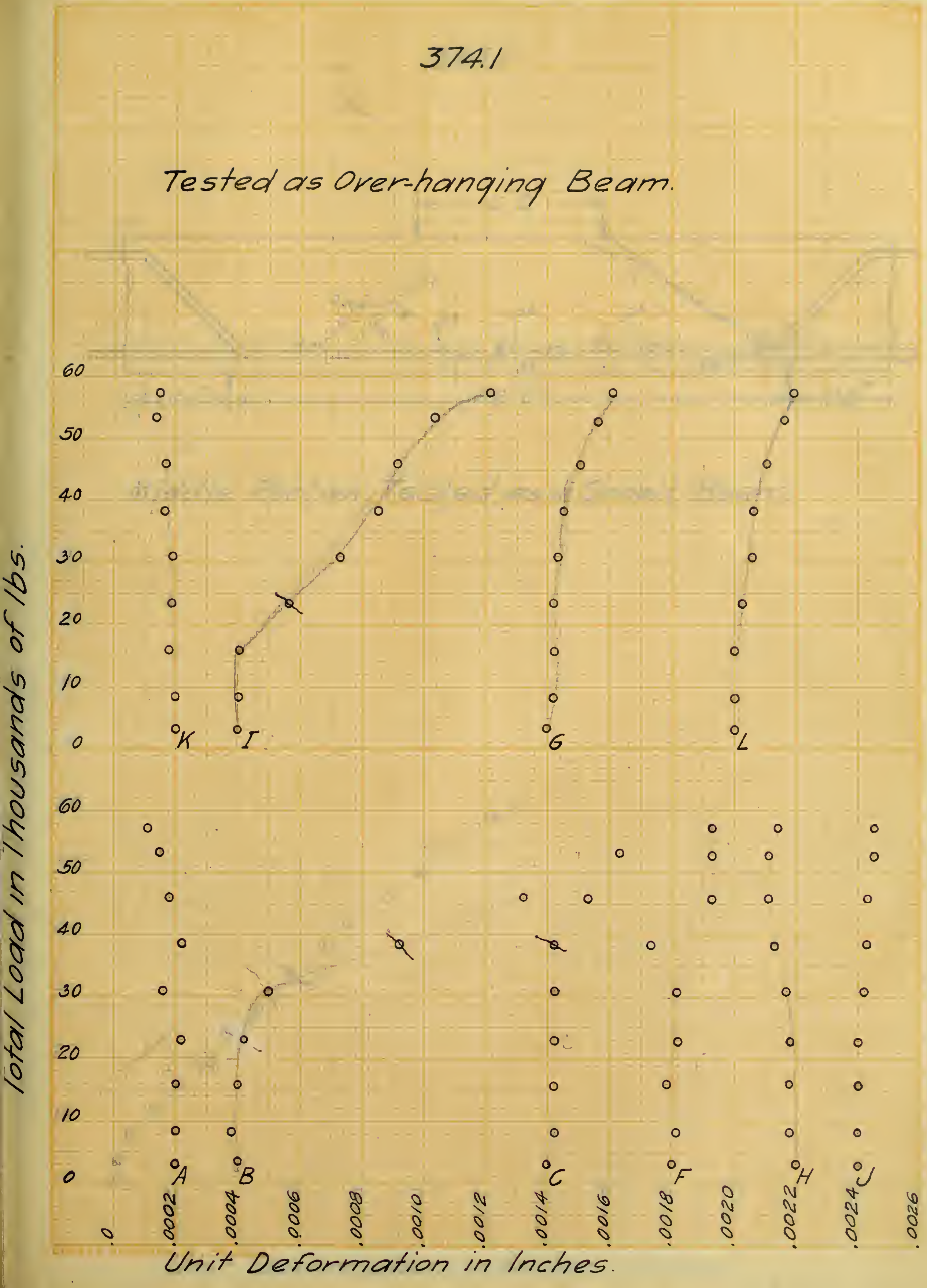

Moment in thousands of inch lbs.
1300
1200
1100
1000
900
800
700
600
500
400
300
200
100
0
Average of L, M, N, & T
Theoretical for L, M, N, & T
Av. of A, B, U, & V
0
5
10
15
20
25
30
35
40
45
50
Stress in thousand lbs.

3762

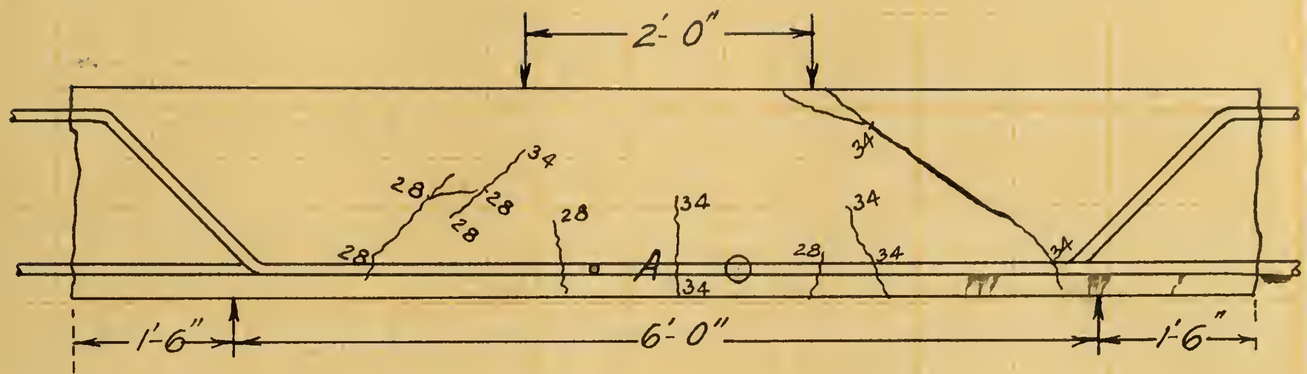

Moment in thousands of inch lbs
Stress in thousand lbs
Theoretical for L,M,N & T
Average of L,M,N & T
Av of A,B,U & V


376.6
Moment in Thousands of Inch lbs.
1400
1300
1200
1100
1000
900
800
700
600
500
400
300
200
100
0
0
5
10
15
20
25
30
35
40
45
50
55
60
Average of I, N, R, & P
Theoretical
Unit Stress in Thousand lbs.

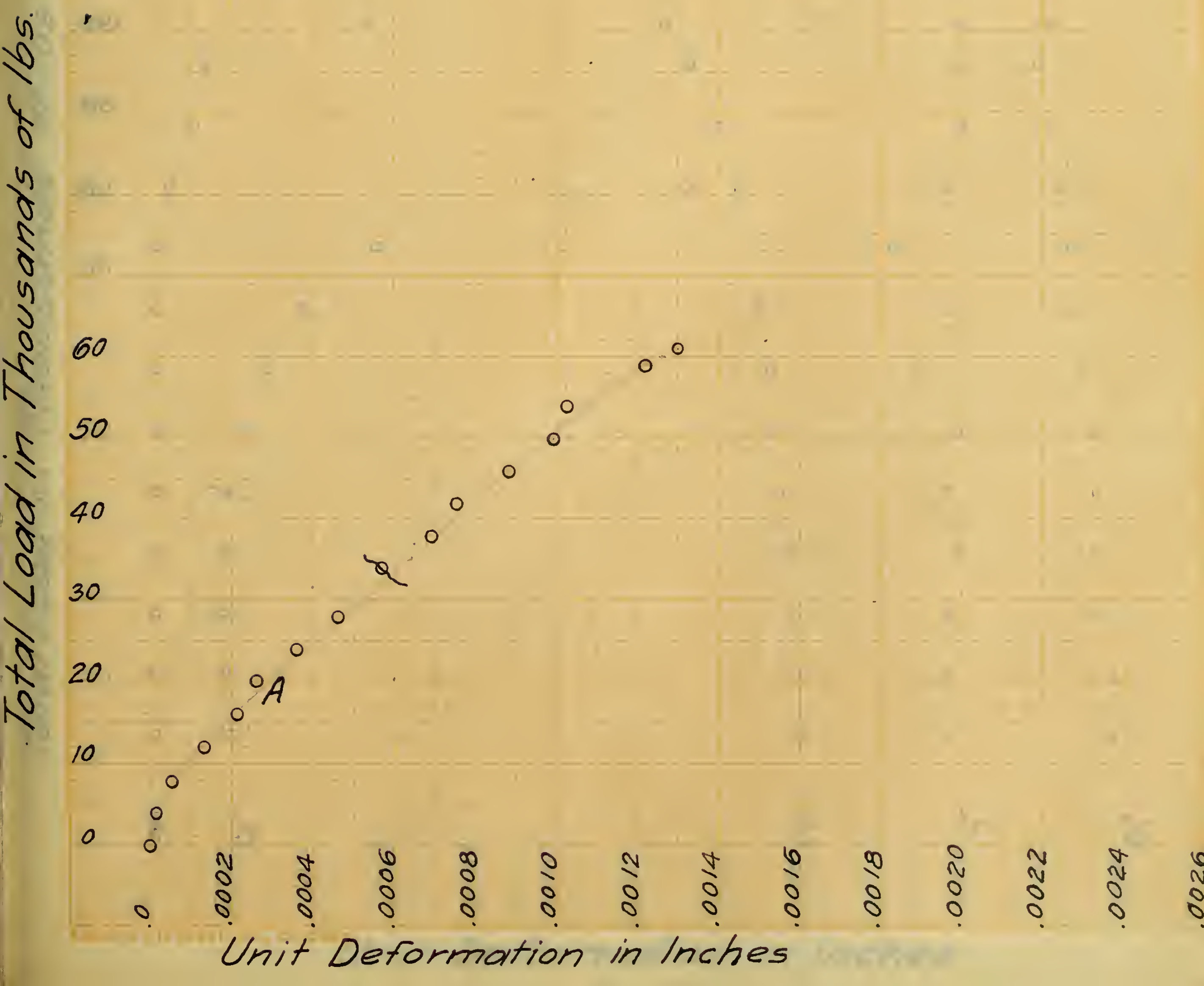
LOAD - DEFORMATION DIAGRAMS

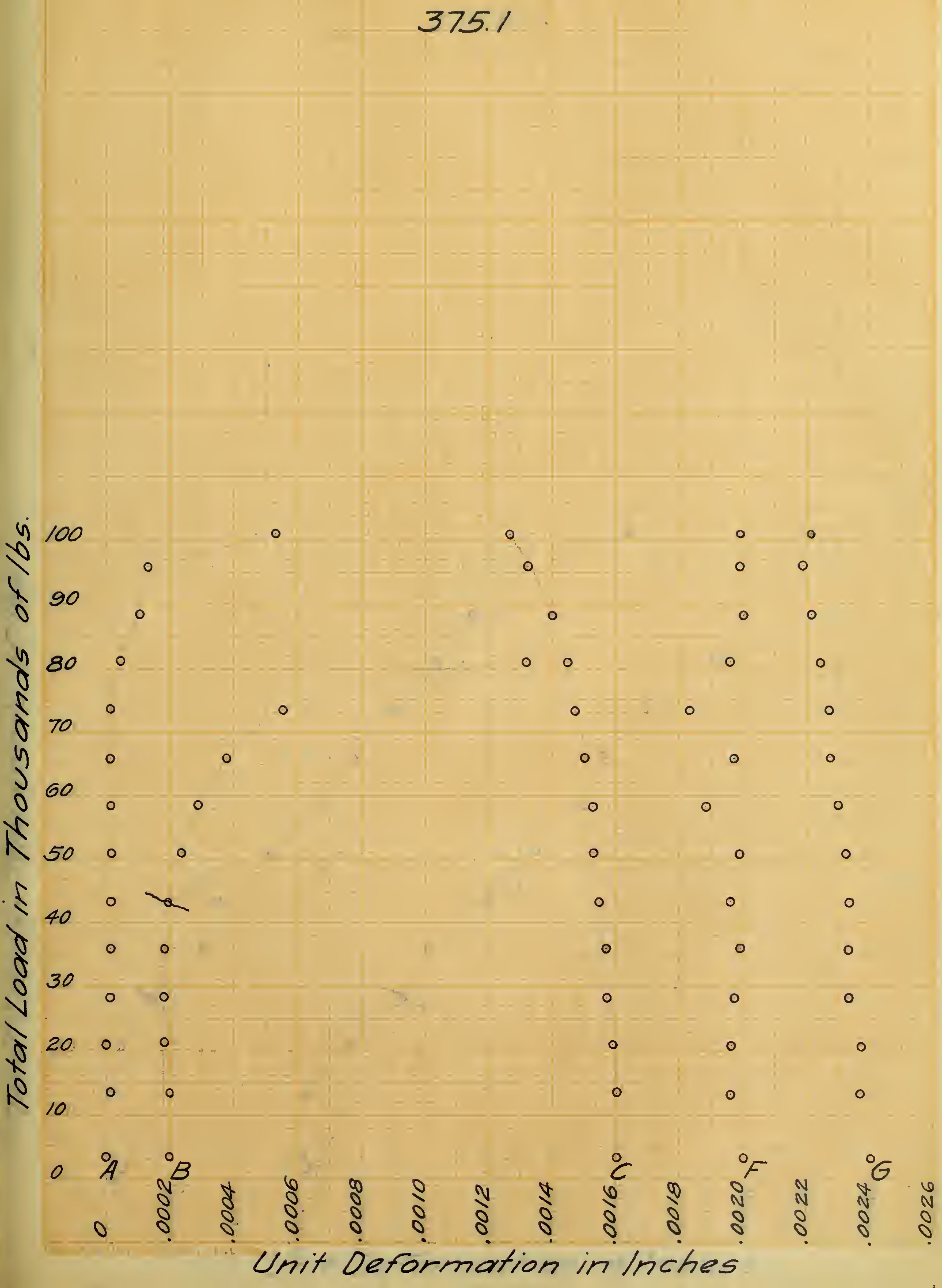

374.1
Tested as Over-hanging Beam.
Total Load in Thousands of lbs.
60
50
40
30
20
10
0
K
I
G
L
60
50
40
30
20
10
0
A
B
C
F
H
J
0
.0002
.0004
.0006
.0008
.0010
.0012
.0014
.0016
.0018
.0020
.0022
.0024
.0026
Unit Deformation in Inches.

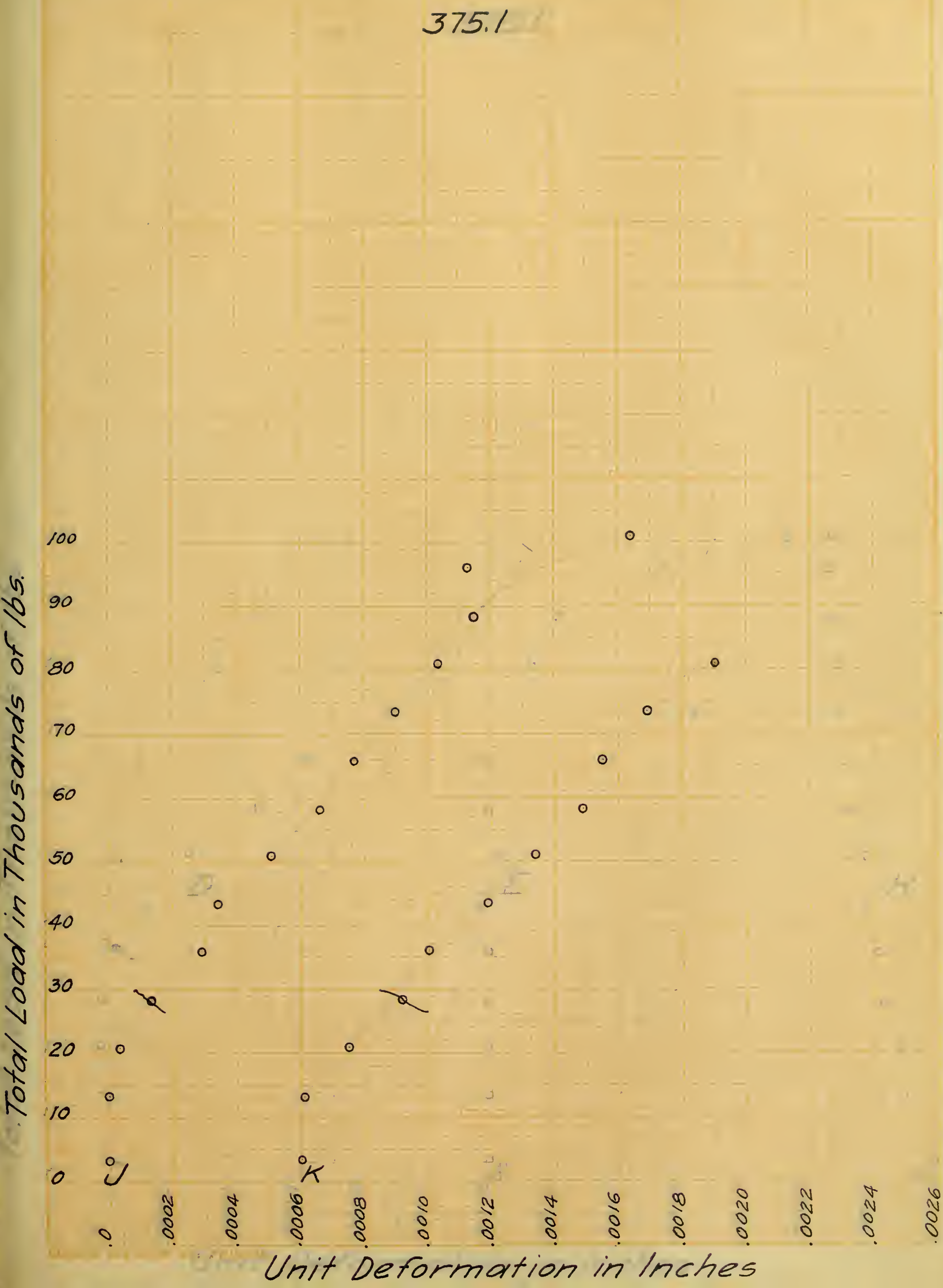
374.1



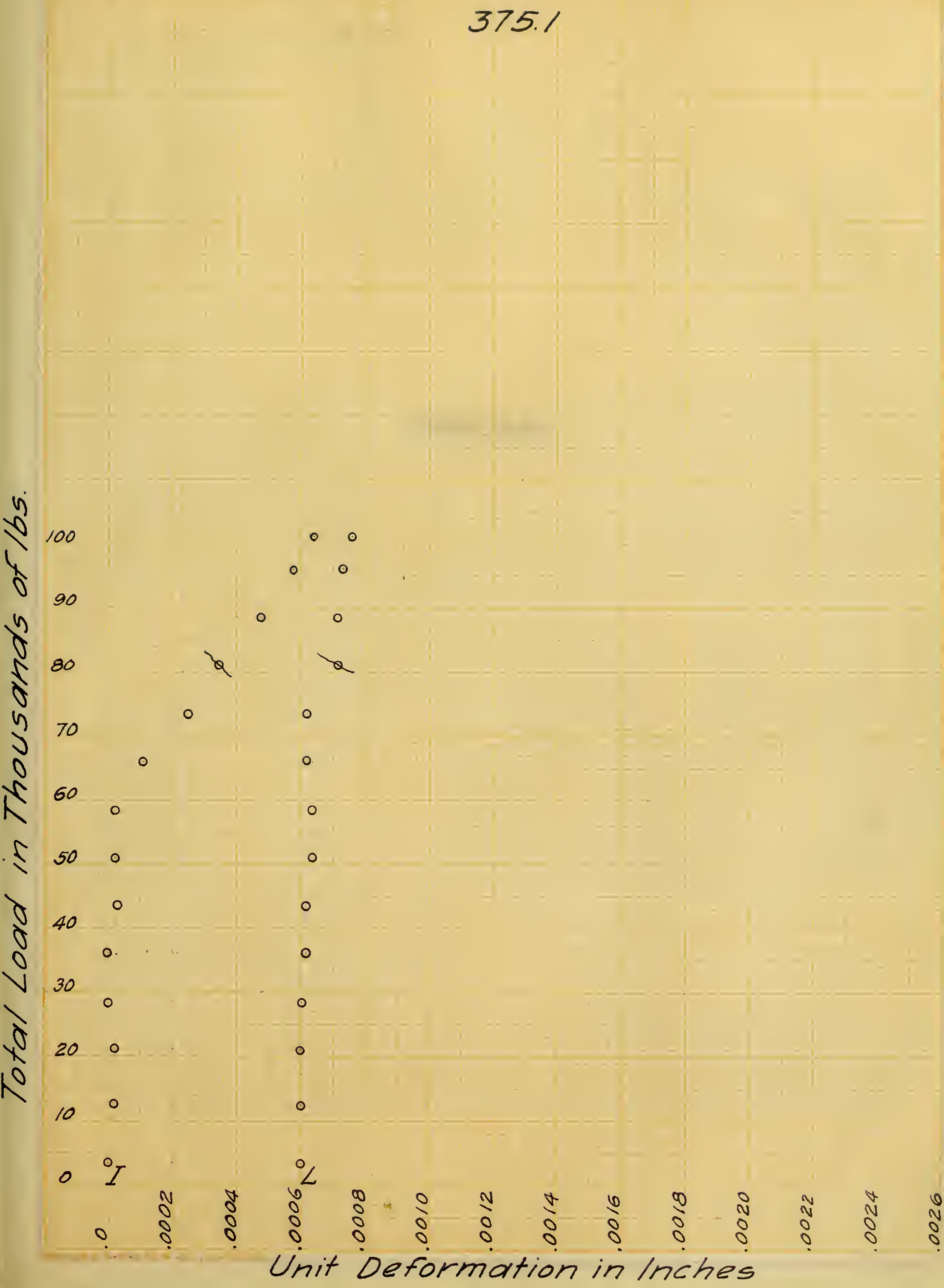
Middle Portion Tested as a Simple Beam.




375.1
Total Load in Thousands of lbs.
100
90
80
70
60
50
40
30
20
10
0
A
B
C
F
G
0
.0002
.0004
.0006
.0008
.0010
.0012
.0014
.0016
.0018
.0020
.0022
.0024
.0026
Unit Deformation in Inches


375.1
100
90
80
70
60
50
40
30
20
10
0
J
K
.0
.0002
.0004
.0006
.0008
.0010
.0012
.0014
.0016
.0018
.0020
.0022
.0024
.0026
Total Load in Thousands of lbs.
Unit Deformation in Inches


375.1
Total Load in Thousands of lbs.
100
90
80
70
60
50
40
30
20
10
0
D
E
H
0.
.0002
.0004
.0006
.0008
.0010
.0012
.0014
.0016
.0018
.0020
.0022
.0024
.0026
Unit Deformation in Inches

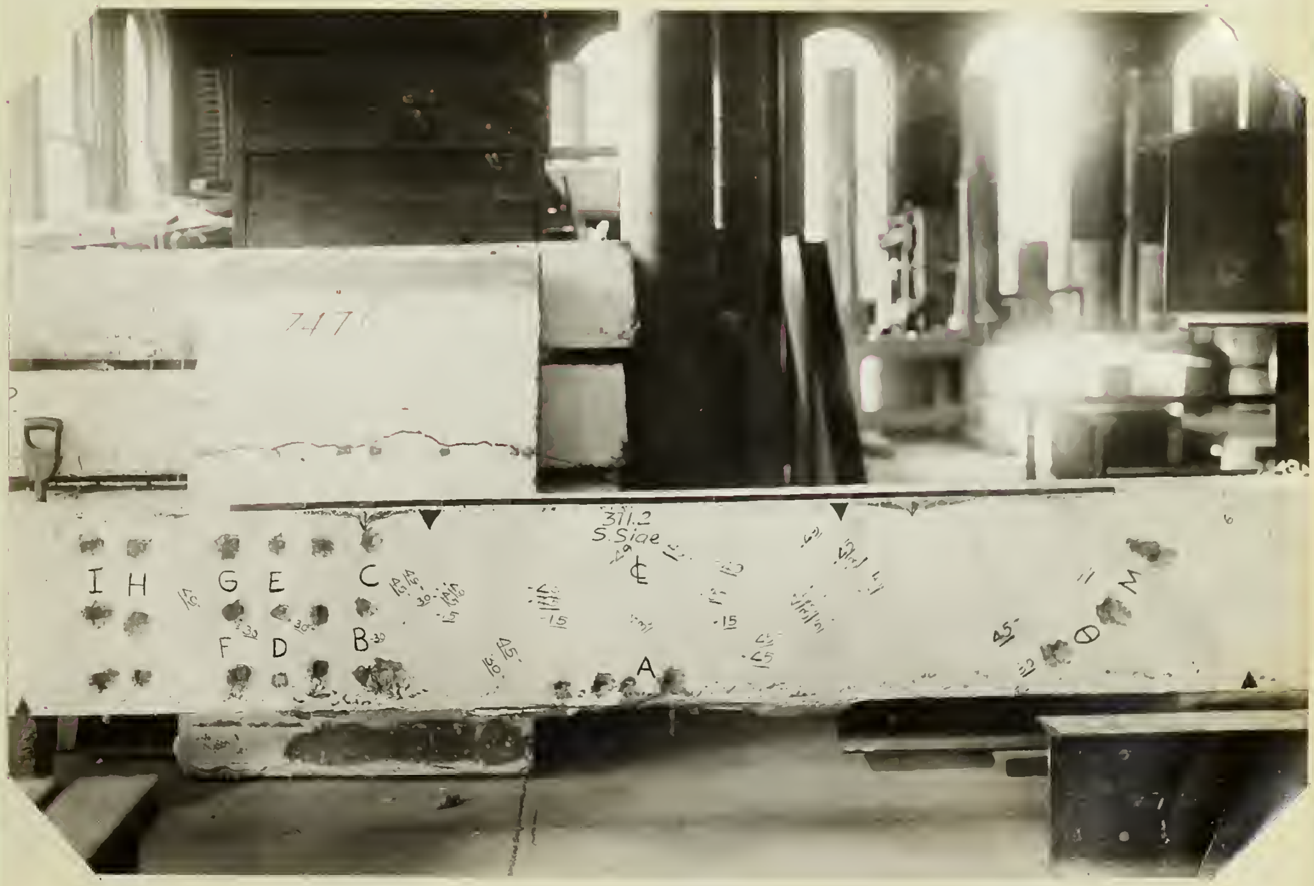


375.1
Total Load in Thousands of lbs.
100
90
80
70
60
50
40
30
20
10
0
I
L
0
.0002
.0004
.0006
.0008
.0010
.0012
.0014
.0016
.0018
.0020
.0022
.0024
.0026
Unit Deformation in Inches

PHOTOGRAPHS

↓ Indicates the load and supports points during test as over-hanging beam.

▼ Indicates the load and support points during test as simple beam.

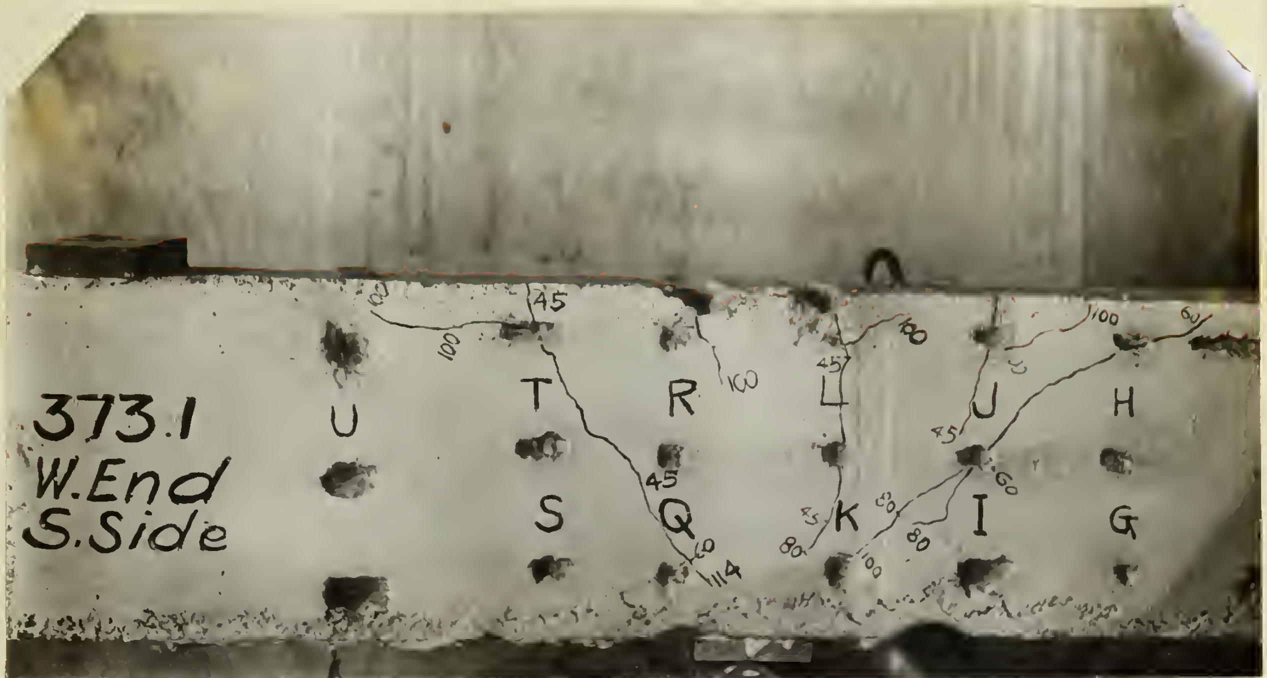


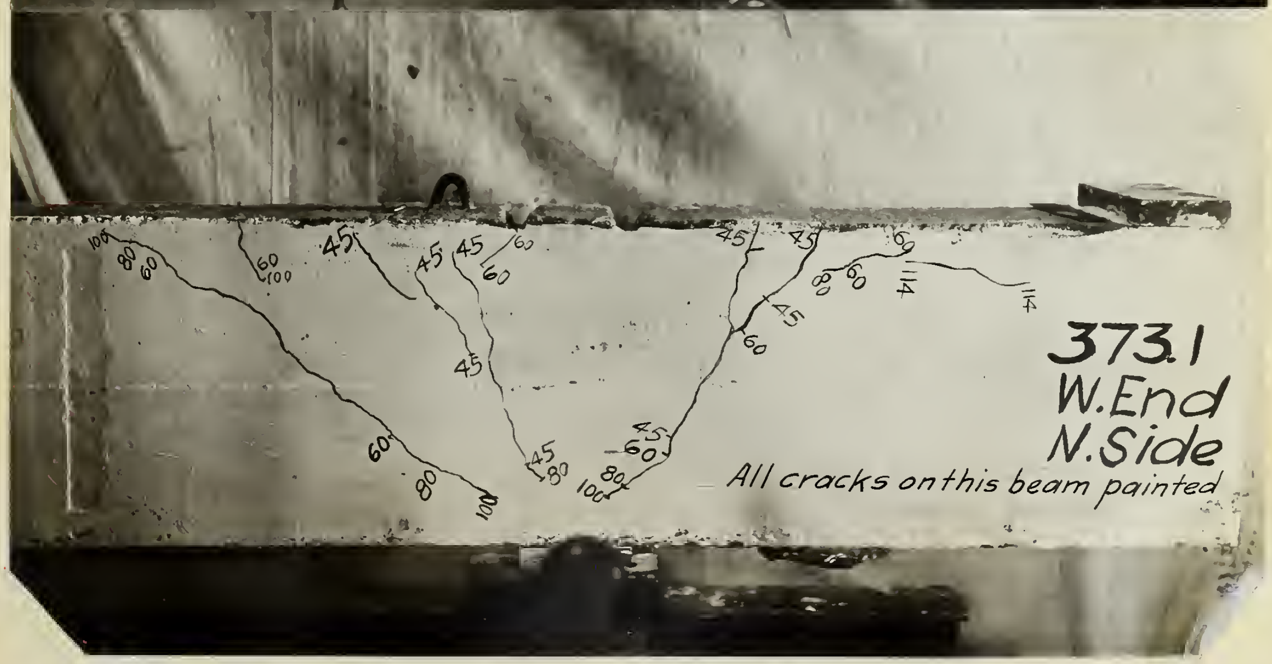
MIDDLE PORTION OF 371.2 AFTER TEST AS SIMPLE BEAM

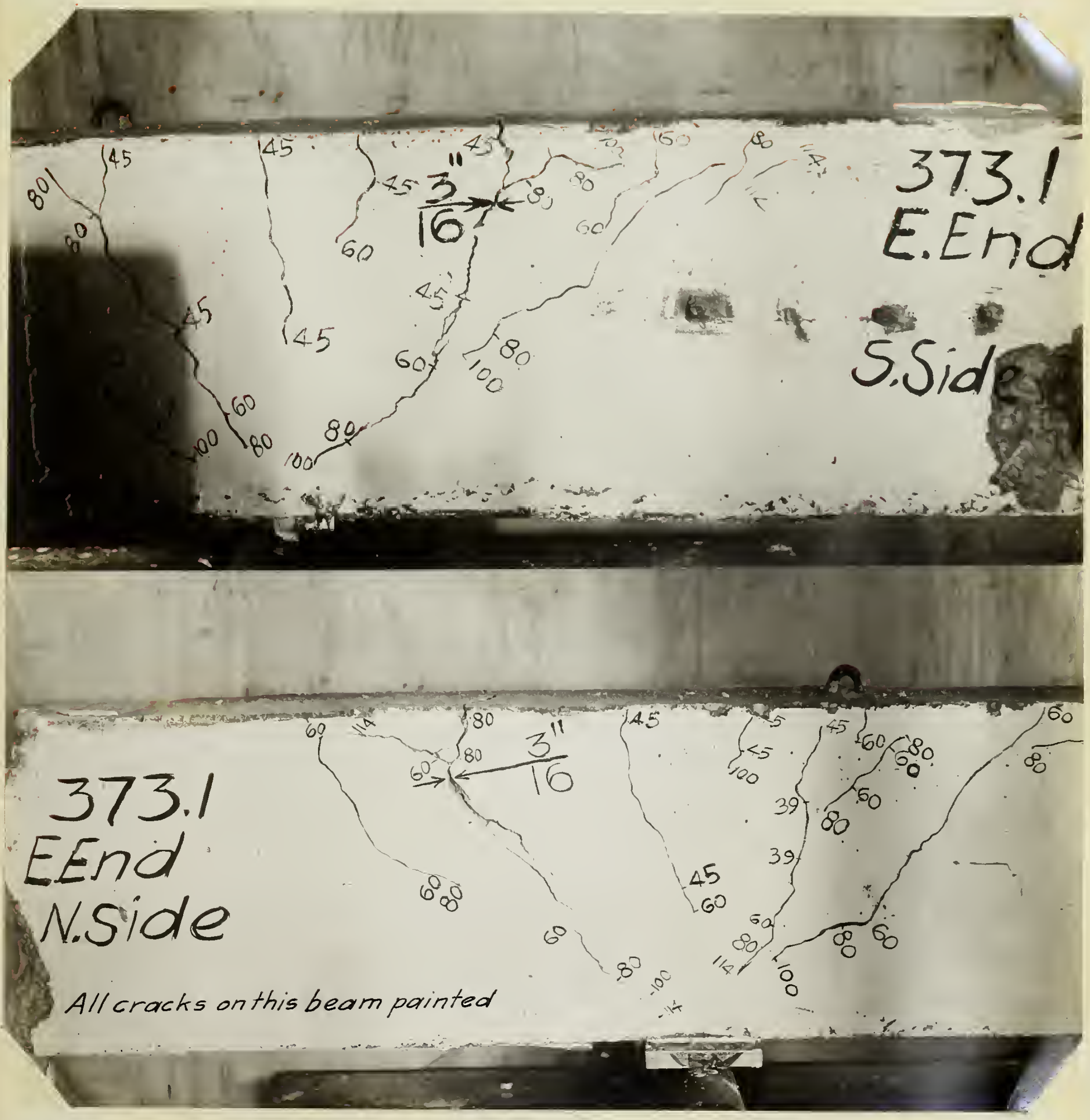
Cracks indicated thus <u>30</u> opened during test as simple beam.

" " " 30 " " " " over-hanging beam.

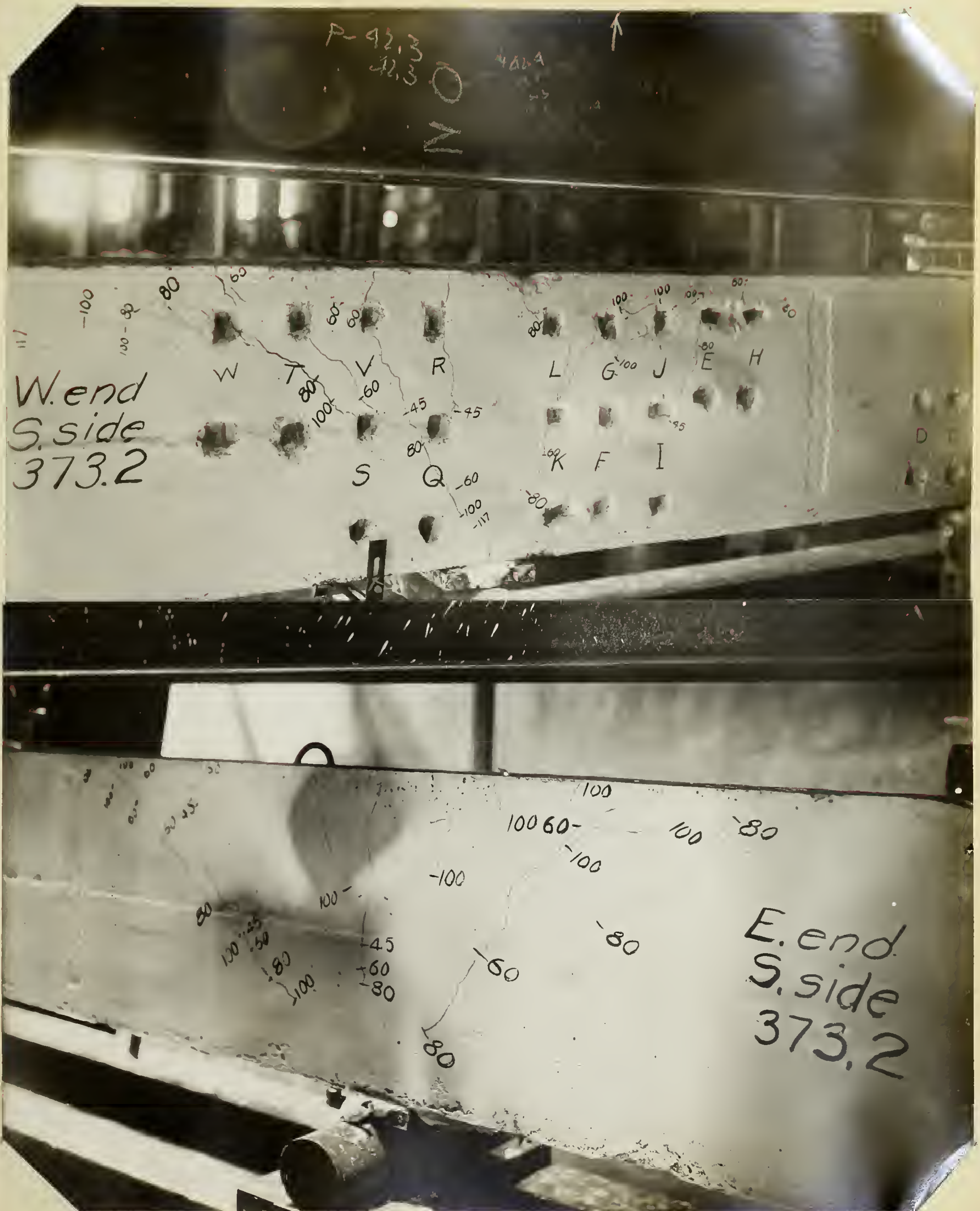

372.1
W. End
N. Side


373.1
W.End
S.Side
U
T
R
L
J
H
S
Q
K
I
G

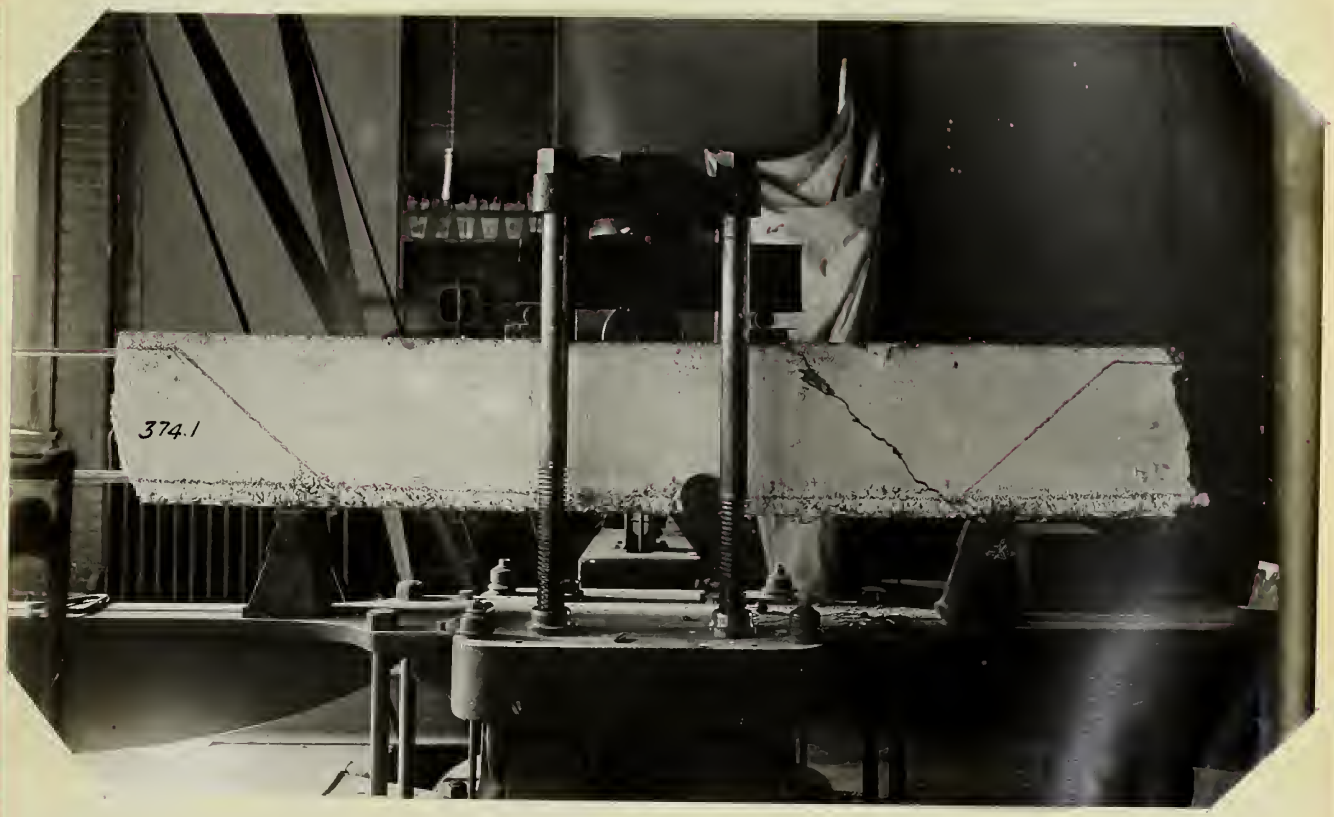

373.1
W.End
N.Side
All cracks on this beam painted

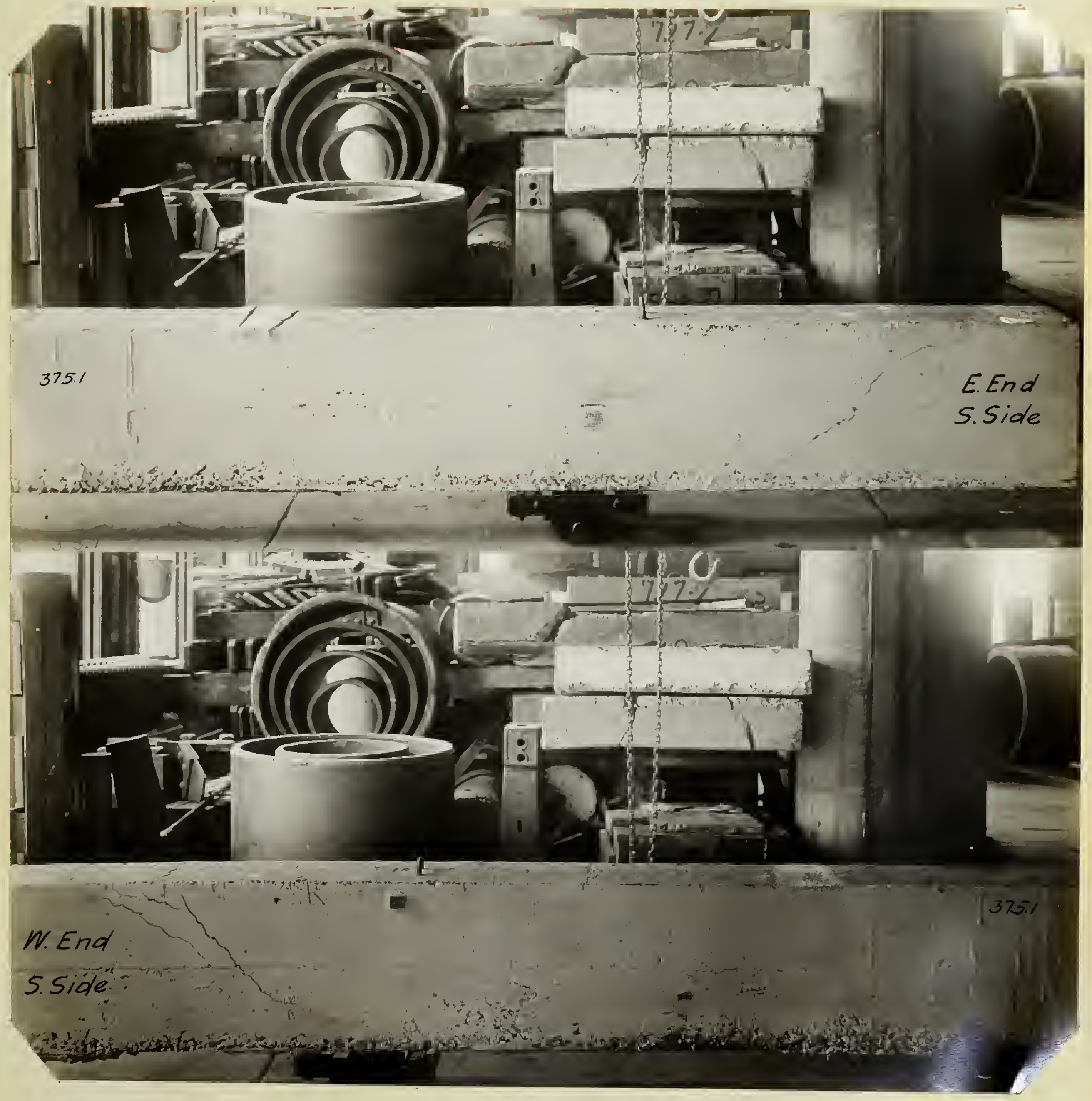


373.1
E.End
S.Side
3"/16
373.1
E.End
N.Side
3"/16
All cracks on this beam painted


W. end
S. side
373.2
W
T
V
R
L
G
J
E
H
S
Q
K
F
I
D
E. end
S. side
373.2

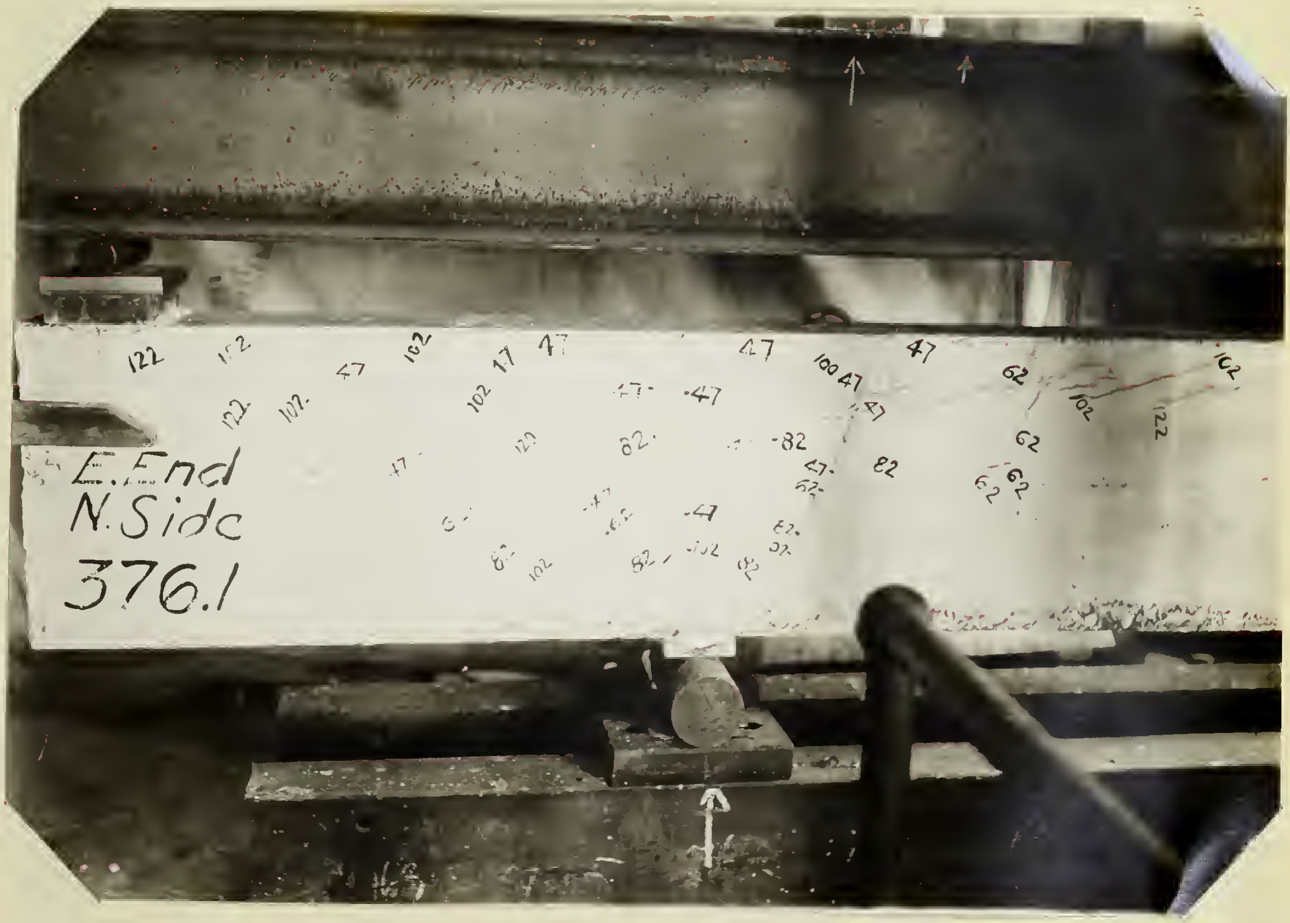

West
End
South Side
374.1
℄


374.1


777-2
375.1
E. End
S. Side
777-2
W. End
S. Side
375.1

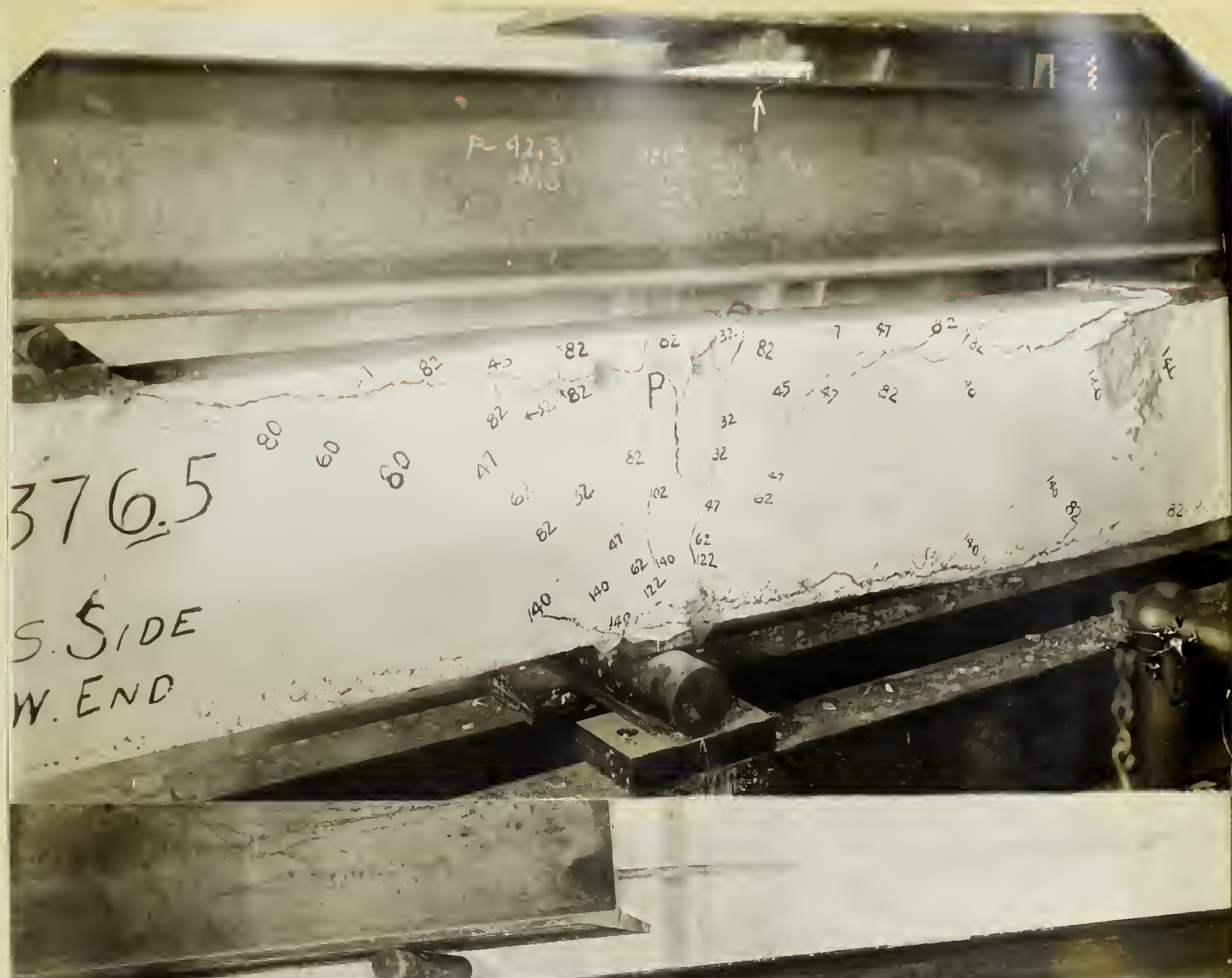

W. End
S. Side
376.1


N. Side
W. End
376.1



E. End
N. Side
376.1


376.2
E.End
S.Side
376.2
W.End
S.Side
376.2
W.End
N Side


376.5
S. SIDE
W. END
P
H
I
J
K
F
G
L
M
N
O 376.5
W. end.
N side


376.6
E.End
S.Side
376.6
W.End
N.Side

SUMMARY SHEET

AND

LOAD - SLIP TABLE

TS.	COMPUTED AND OBSERVED DATA								BEAM NO.
T STR. NCH BES	LONGITUDINAL REINFORCEMENT				COMP. BOND STRESS	SHEAR		BOND AT FIRST SLIP	
	f_s AT SUPPORT		f_s AT CENTER			$\frac{V}{bd}$	$\frac{V}{bjd}$		
	COMPUTED	OBSERVED	COMPUTED	OBSERVED					
550	30300	31600	7400	2500	256	131	152		371.2
	Load 45000								
030	34400	31600	35000	35000+	222	226	252	170	372.1
	Load 100000								
230	34900	29800	33300	36000	212	216	241	170•	372.2
	Load 100000								
130	36000	36000	18000	17800	229	246	274	202	373.1
	Load 100000								
580	34900	31600	17200	13000	237	242	270	96•	373.2
	Load 100000								
530°					234	119	135		374.1
716°					223	208	238		375.1
520°	42200	42900	30500	34800	290	296	339	100•	376.1
	Load 120000								
020	57200	51300	41500	44200	372	380	435	127•	376.2
	Load 160000								
500°	42000	33100	21700	21000	286	291	336	167•	376.5
	Load 120000								
510	55500	54100	29800	33000	367	375	432	170	376.6
	Load 160000								
			33300	33000	193	191	227		371.2
			Load 45000						
			26500	28500	249	254	293		374.1
			Load 50000						

•...oad slipped.

TABLE V

BEAM NO.	SECTION CENTER	SECTION SUPPORT	AGE	MAXIMUM LOAD POUNDS	LONGITUDINAL REINFORCEMENT KIND	BARS CTR.	BARS SUP.	PER CENT CTR.	PER CENT SUPPORT TOP	PER CENT SUPPORT BOT.	RATIO CTR./SUP.	YIELD POINT	WEB REINFORCEMENT KIND	SIZE	SPACED	PER CENT	YIELD POINT	AUXIL. TESTS M. OF R. 6 × 8 IN BEAMS	AUXIL. TESTS UNIT STR. 6 INCH CUBES	f_s AT SUPPORT COMPUTED	f_s AT SUPPORT OBSERVED	f_s AT CENTER COMPUTED	f_s AT CENTER OBSERVED	COMP. BOND STRESS	SHEAR $\frac{V}{bd}$	SHEAR $\frac{V}{bjd}$	BOND AT FIRST SLIP	BEAM NO.
371.2	16"	15½"	62	65 400	Plain ϕ	4-¾	2-¾	1.44	.72	.00	2.0		Plain ϕ U Stirrups	¼	4	.31		303	2550	30 300 Load	31 600 45 000	7400	2 500	256	131	152		371.2
372.1	15½	15¾	68	110 000	do.	2-¾	4-¾	.72	1.44	.00	.50	33 200	do	¼	4	.31		338	2030	34 400 Load	31 600 100	35 000 000	35000+	222	226	252	170	372.1
372.2	16¼	15½	64	103 600	do.	2-¾	4-¾	.72	1.44	.00	.50		do	¼	4	.31	38 600	318	2230	34 900 Load	29800 100	33300 000	36000	212	216	241	170•	372.2
373.1	15	15	65	114 000	do.	4-¾	4-¾	1.44	1.44	.00	1.00	34 400	do	¼	4	.31		244	2130	36 000 Load	36000 100	18000 000	17 800	229	246	274	202	373.1
373.2	15¾	15½	60	116 000	do.	4-¾	4-¾	1.44	1.44	.00	1.00	34 200	do	¼	4	.31	37 800	310	2680	34900 Load	31.600 100	17200 000	13 000	237	242	270	96•	373.2
374.1	15	15¼	61	57 100	do.	4-¾	2-¾ 2-¾	1.48	.74	.74	2.00	35 100	None			.00		254	1630					234	119	135		374.1
375.1	15	15½	73	102 100	do.	2-¾ 2-½	2-¾ 2-⅝ 2-½	1.06	1.25	.32	.85	60 500	Plain ϕ Rigidly Attached	¼	4	.31		440	1716°					223	208	238		375.1
376.1	14¼	15	67	141 100	Corr. ϕ	2-¾ 2-½	4-¾ 2-½	1.13	1.47	.32	.77		See Page 14	.21		.44		294	1520°	42200 Load	42900 120	30500 000	34800	290	296	339	100•	376.1
376.2	14	14¾	67	178 000	do	2-¾ 2-½	4-¾ 2-½	1.13	1.47	.32	.77	56 600	See Page 14	.21		.44	41 350	265	2020	57200 Load	51300 160	41 500 000	44200	372	380	435	127•	376.2
376.5	14½	15	61	139 800	do.	4-¾	4-¾ 2-¾	1.44	1.45	.73	1.00		See Page 14	¼		.75			1600°	42 000 Load	33 100 120	21700 000	21000	286	291	336	167•	376.5
376.6	14	15	64	180 000	do.	4-¾	4-¾ 2-¾	1.44	1.45	.73	1.00	54 800	See Page 14	¼		.75	60 200	436	2510	55500 Load	54100 160	29800 000	33000	367	375	432	170	376.6
371.2	2nd Test 9 Ft. span.		63	49 000	Plain ϕ	4-¾		1.44														33300 Load	33000 45 000	193	191	227		371.2
374.1	2nd Test 6 Ft. span.		67	61 000	do	4-¾		1.48														26 500 Load	28500 50 000	249	254	293		374.1

° Values of cube strength low – see page 30   • Value of bond when one rod only had slipped.

OF SLIP OF LONGITUDINAL RODS

LOAD 100 000			LOAD 120 000			LOAD 140 000			LOAD 160 000			LOAD 180 000		
T	B	S	T	B	S	T	B	S	T	B	S	T	B	S
		45												
13.4	158	72												
13.9	400	109												
11.9	340	72												
12.8	155	1082												
13.2	159	1078												
		7												
		60												
18.2	212	32	18.3	216	370									
16.8	197	31	17.1	200	353									
		10												
15.2	179	30												
14.9	175	17												
13.0	153	242	15.2	179	450	No Reading								
11.7	418	242	13.5	482	450	"	"							
18.5	218	143	21.4	252	310	"	"							
		102			246	"	"							
		214			233	"	"							
			13.7	442	12	15.9	512	42	18.7	603	115	No Reading		1210
			10.7	345	11	12.6	407	52	14.2	458	152	"	"	1280
9.9	116	17	12.0	141	41	14.5	170	130	16.4	193	328	"	"	800
8.0	258	17	8.6	278	41	10.0	322	130	11.5	370	328	"	"	800
			12.2	394	9	13.0	420	112	12.9	416	351			1470
			18.0	212	9	20.2	238	112	22.0	259	351			1470
		175			430			158						
		270			570									
10.7	324	12	8.8	267	31	12.5	380	60	12.1	367	100	14.3	433	167
17.0	200	12	20.4	240	31	23.0	270	60	—	—	100	—	—	167
7.5	227	5	11.5	348	7	13.7	415	13	15.9	482	60	16.9	512	350
12.3	145	5	14.6	172	7	19.5	230	13	19.7	232	60	—		350
8.4	322	5	10.0	385	9	12.9	495	22	16.3	627	50	15.4	592	175
									10.3	396	28	10.8	415	135

T = Total Measured Tension on Rod

$$B = \text{Unit Bond} = \frac{\text{Total Tension on Rod}}{\text{Imbedded Area}}$$

S = Total Amount of Slip at the Particular Stage of Loading Expressed in Ten-Thousandths of an Inch.

TABLE No. VI SHOWING AMOUNT AND NATURE OF SLIP OF LONGITUDINAL RODS

BEAM NO	KIND OF RODS	DIAM. RODS	GAGE USED	INDEX No.	INITIAL SLIP							LOAD 60 000			LOAD 80 000			LOAD 100 000			LOAD 120 000			LOAD 140 000			LOAD 160 000			LOAD 180 000		
					UNIT TENSION ON ROD	TOTAL TENSION	LENGTH IMBEDDED	BOND	BOND V/mod	LOAD	AM'T SLIP	T	B	S	T	B	S	T	B	S	T	B	S	T	B	S	T	B	S	T	B	S
372.1	Plain			1						80.0	.0007									45												
			H	2	25.9	11.4	36	61	170	80.0								13.4	158	72												
	Round		R	4	16.2	7.2	15	202	170	80.0	7							13.9	400	109												
			P	2	17.0	7.9	15	226	170	80.0	9							11.9	340	72												
372.2			J	1	28.0	12.4	35	150	170	80.0	8							12.8	155	1082												
	PLAIN		I	2	24.0	10.6	35	129	170	80.0	18							13.2	159	1078												
				3						80.0	1									7												
	ROUND			4						100.0	4																					
373.1				3						86.0										60												
	PLAIN		N	2	32.2	14.2	36	168	170	80.0								18.2	212	32	18.3	216	370									
	ROUND		O	1	38.0	16.8	36	198	202	100.0	31							16.8	197	31	17.1	200	353									
				4						98.0										10												
373.2			O	1	13.4	5.9	36	7.0	96	45.0	2	8.6	100	5	12.1	142	11	15.2	179	30												
	PLAIN		N	2	28.0	12.4	36	146	170	80.0	7							14.9	175	17												
	ROUND			3		Practically No Slip																										
				4		"			"		"																					
376.1			L	2	17.9	7.9	36	93	100	47.3	3	9.8	115	30	13.1	154	105	13.0	153	242	15.2	179	450	No Reading								
	CORR.		S	2	11.3	5.0	12	177	100	47.3	3	8.4	300	30	10.5	375	105	11.7	418	242	13.5	482	450	"	"							
		All 3/4" φ	T	1	30.0	13.2	36	156	150	70.0					15.0	176	42	18.5	218	143	21.4	252	310	"	"							
	ROUND			3						78.0							18			102			246	"	"							
				4						78.0							80			214			233	"	"							
376.2			W	3	26.1	11.5	13	377	202	100.0	5										13.7	442	12	15.9	512	42	18.7	603	115	No Reading		1210
			X	4	18.7	8.3	13	270	202	100.0	3										10.7	345	11	12.6	407	52	14.2	458	152	"	"	1280
	CORR.		L	2	7.2	3.2	36	37	96	45.0	2	4.8	56	4	7.9	93	6	9.9	116	17	12.0	141	41	14.5	170	130	16.4	193	328	"	"	800
			S	2			13		96	45.0	2	2.7	89	4	5.8	170	6	8.0	258	17	8.6	278	41	10.0	322	130	11.5	370	328	"	"	800
	ROUND		Y	1	22.4	9.9	13	324	202	100.0	8										12.2	394	9	13.0	420	112	12.9	416	351			1470
			T	1	32.7	14.4	36	170	202	100.0	8										18.0	212	9	20.2	238	112	22.0	259	351			1470
376.5	CORR.			3																175			430			158						
	ROUND			4													15			270			570									
376.6			S	1	12.4	5.5	14	171	170	80.0	5							10.7	324	12	8.8	267	31	12.5	380	60	12.1	367	100	14.3	433	167
			I	1	31.1	13.8	36	162	170	80.0	5							17.0	200	12	20.4	240	31	23.0	270	60	—	—	100	—	—	167
	CORR.		V	2	15.5	6.9	14	208	170	80.0	2							7.5	227	5	11.5	348	7	13.7	415	13	15.9	482	60	16.9	512	350
			P	2	18.1	8.0	36	94	170	80.0	2							12.3	145	5	14.6	172	7	19.5	230	13	19.7	232	60	—		350
	ROUND		W	3	15.0	6.6	11	256	170	80.0	2							8.4	322	5	10.0	385	9	12.9	495	22	16.3	627	50	15.4	592	175
			X	4	21.5	9.5	11	365	300	140.0	9																10.3	396	28	10.8	415	135
374.1	PLAIN			1						53.5		At 57 100			Slip = 0.0004																	
				2						53.5		"			"		"															
	ROUND			3						53.5	10																					
375.1	PLAIN			1 & 2		At 102 100 lbs Load						Slip = 0.1900																				
	ROUND			3 & 4		"		"			"	"	= 0.0630																			

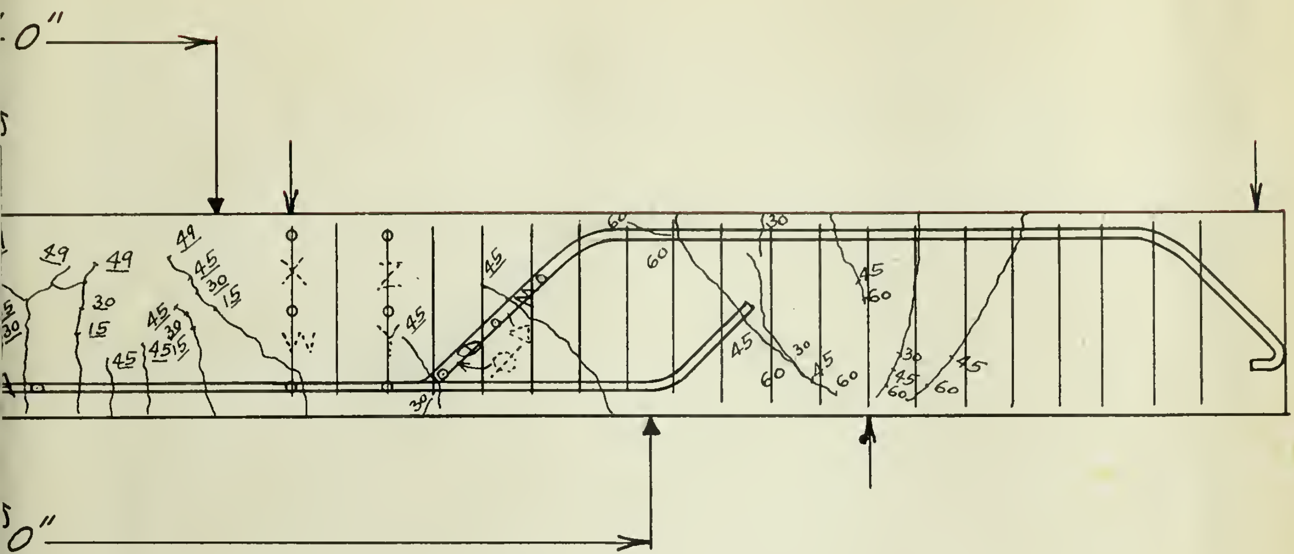
T = Total Measured Tension on Rod

B = Unit Bond = $\frac{\text{Total Tension on Rod}}{\text{Imbedded Area}}$

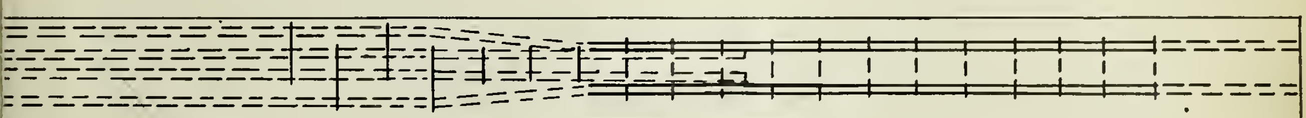
S = Total Amount of Slip at the Particular Stage of Loading Expressed in Ten-Thousandths of an Inch.

SKETCHES OF BEAMS

71.2



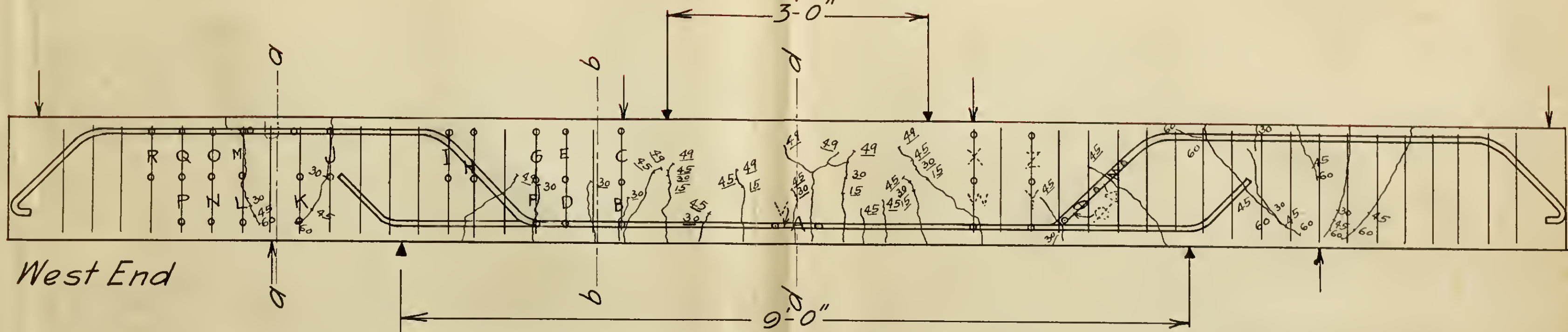
er supports, the middle
above indicated. All figures
h indicate cracks formed during test



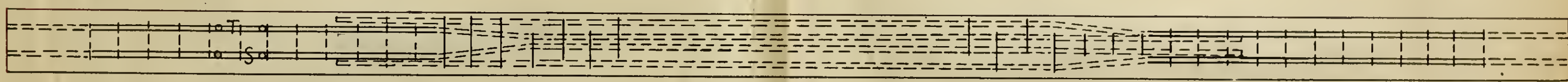
on d-d

7

371.2



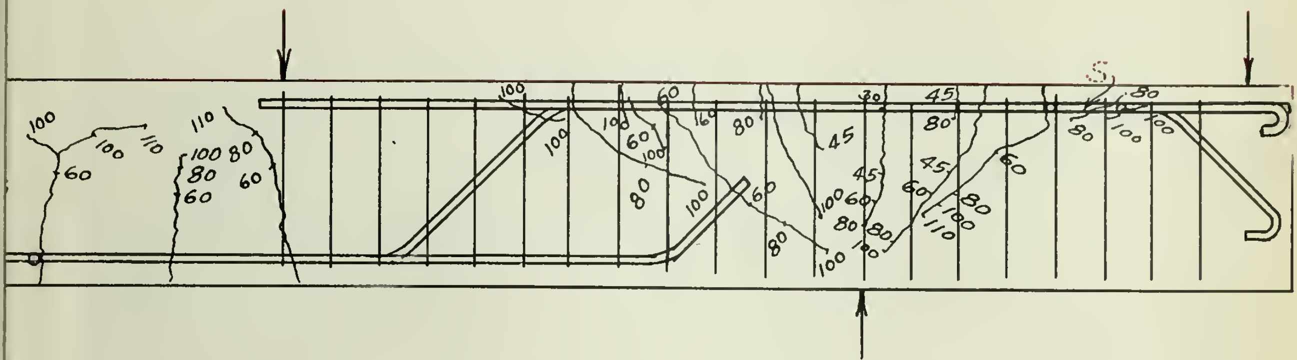
Note: After failure over supports, the middle portion was tested as above indicated. All figures with a dash underneath indicate cracks formed during test as a simple beam.

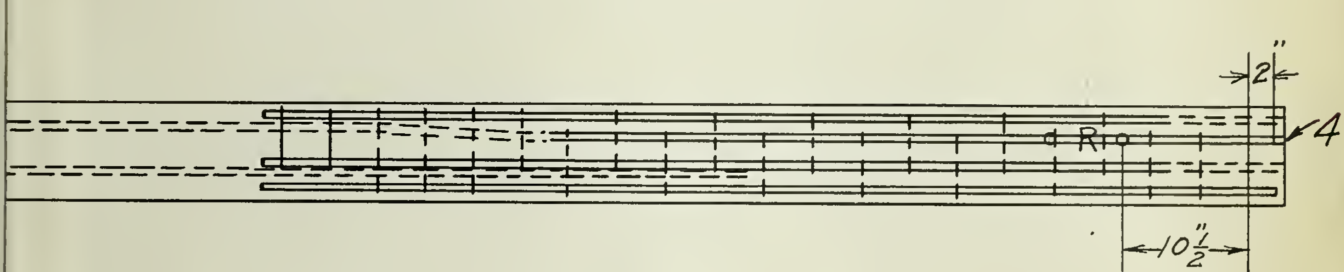


Section a-a Section b-b Section d-d

Fig. 7

72.1





ion d-d

8

372.1

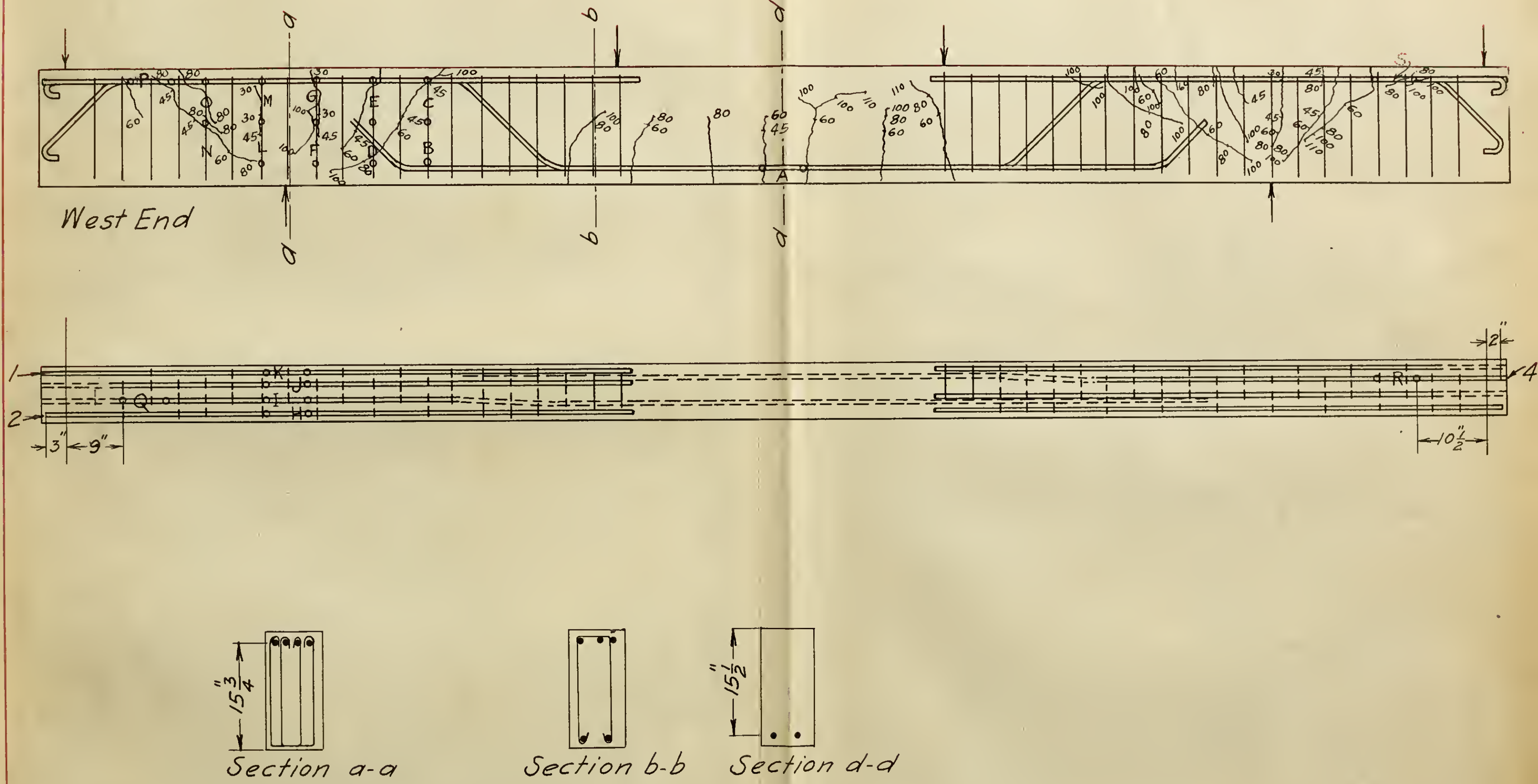
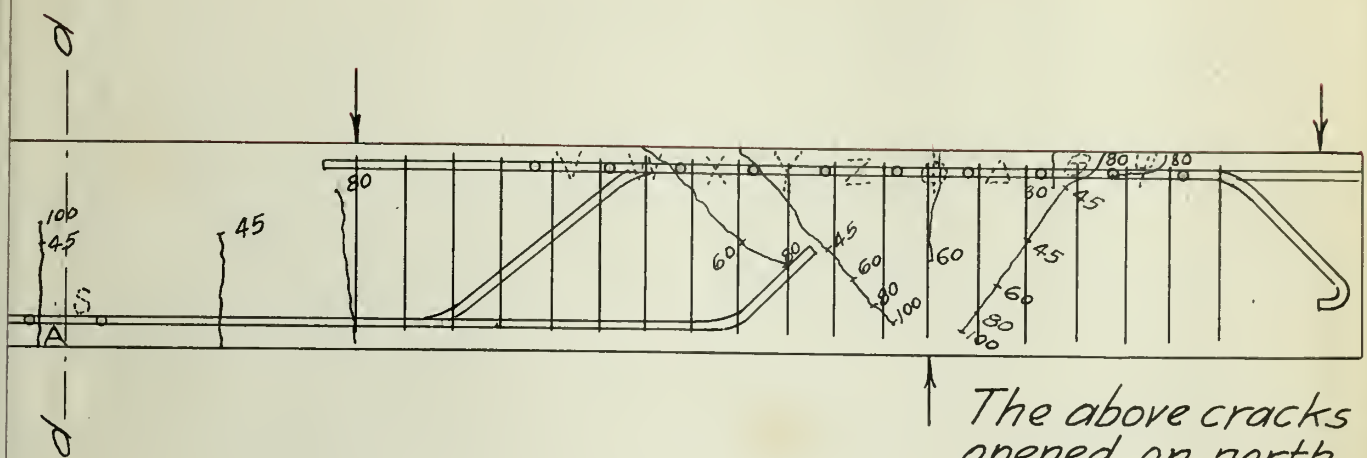
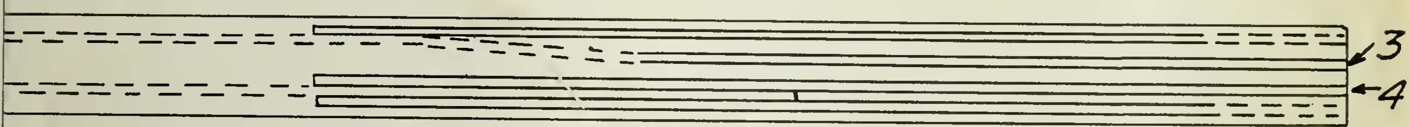
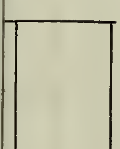

West End
a
b
d
A
B
C
D
E
F
G
L
M
N
O
P
Q
R
S
H
I
J
K
1
2
4
3"
9"
2"
10½"
15¾"
15½"
Section a-a
Section b-b
Section d-d

Fig. 8



The above cracks opened on north face.





ection d-d

. 9

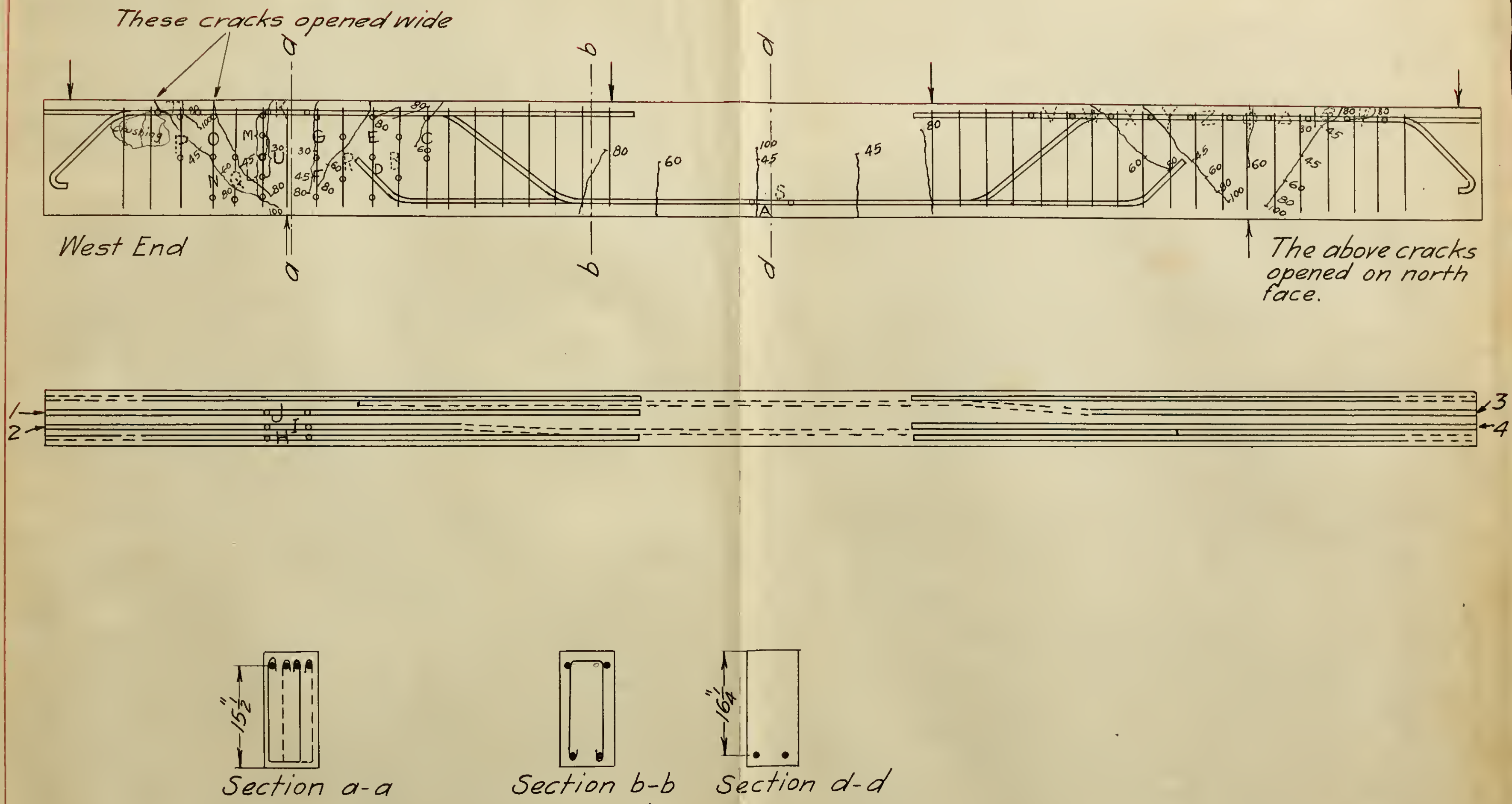
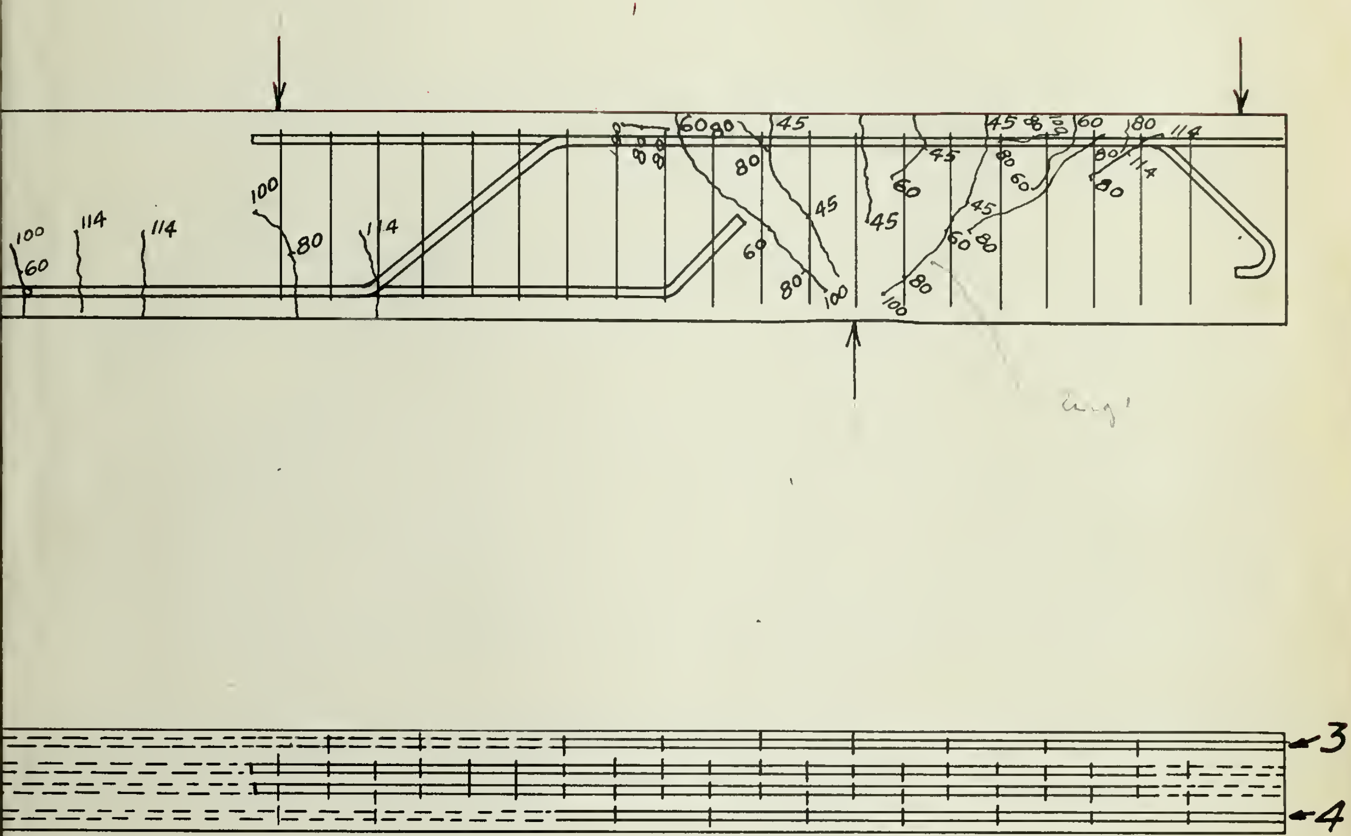

These cracks opened wide
West End
The above cracks opened on north face.
Crushing
a
b
d
A
1
2
3
4
15½"
Section a-a
Section b-b
16¼"
Section d-d

Fig. 9

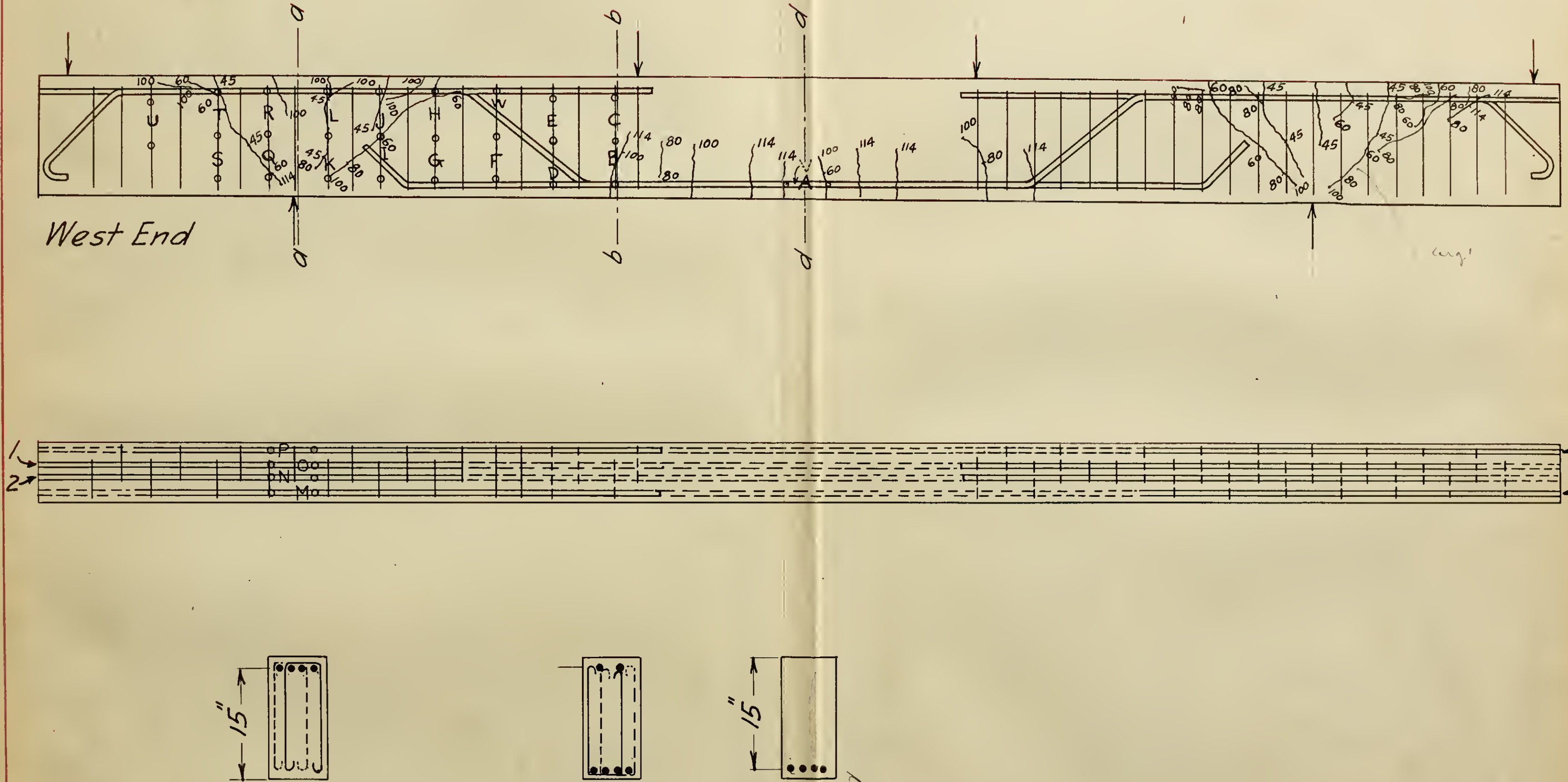
73.1



on d-d

10

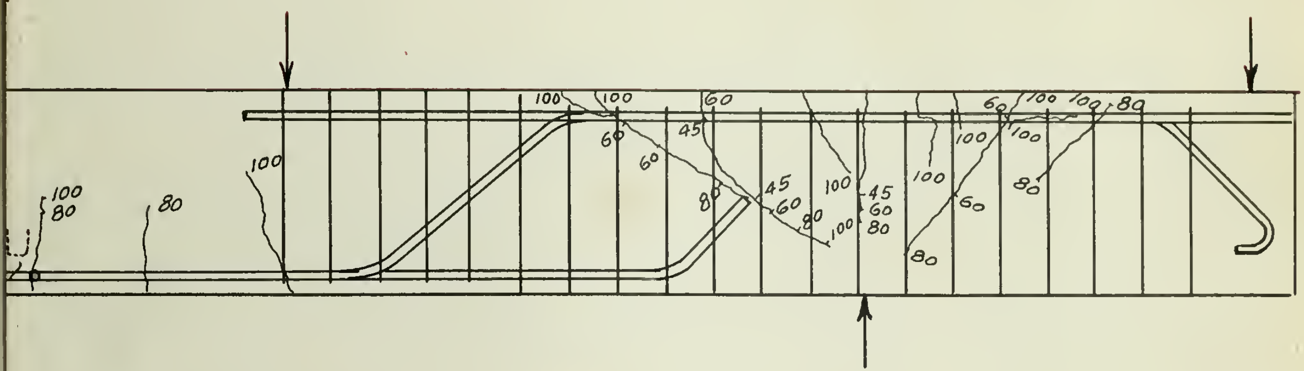
373.1

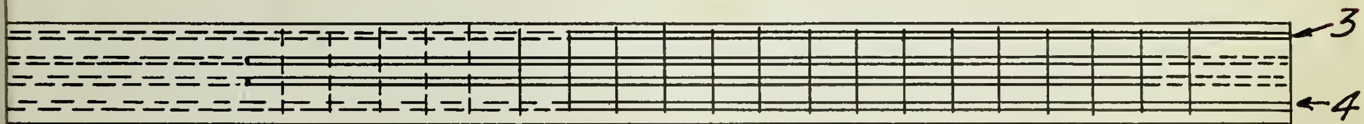


Section a-a   Section b-b   Section d-d

Fig. 10

3.2





ion d-d

11

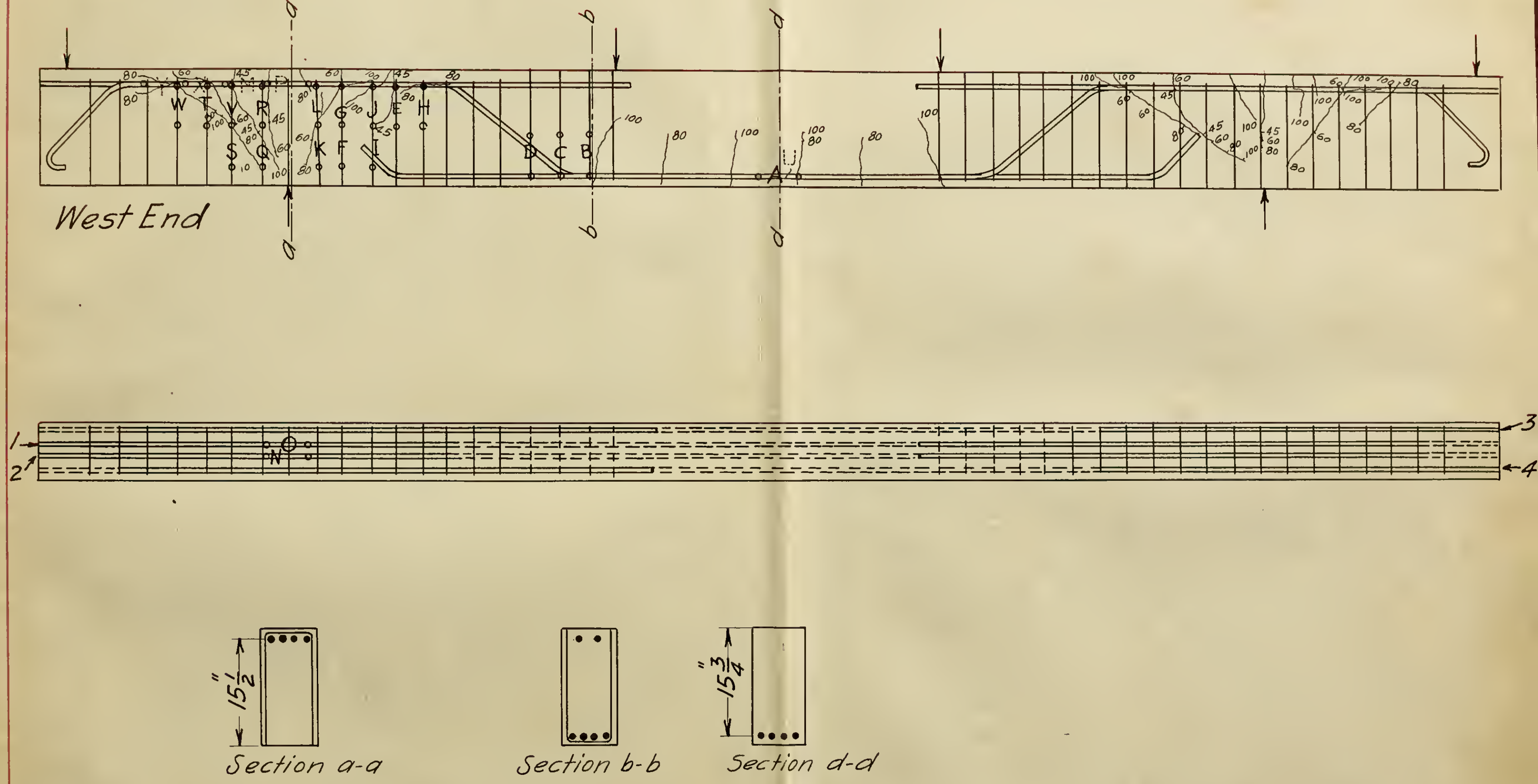
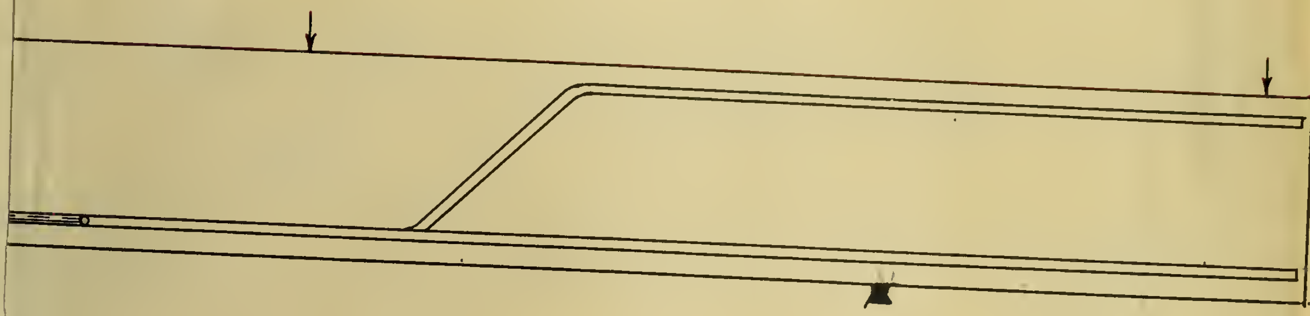
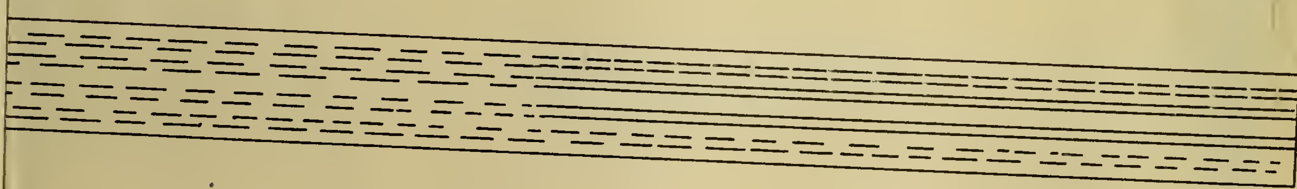

West End
a-a
b-b
d-d
W T V R L G J E H
S Q K F I
D C B
A
N
1
2
3
4
15 1/2"
15 3/4"
Section a-a
Section b-b
Section d-d

Fig. 11





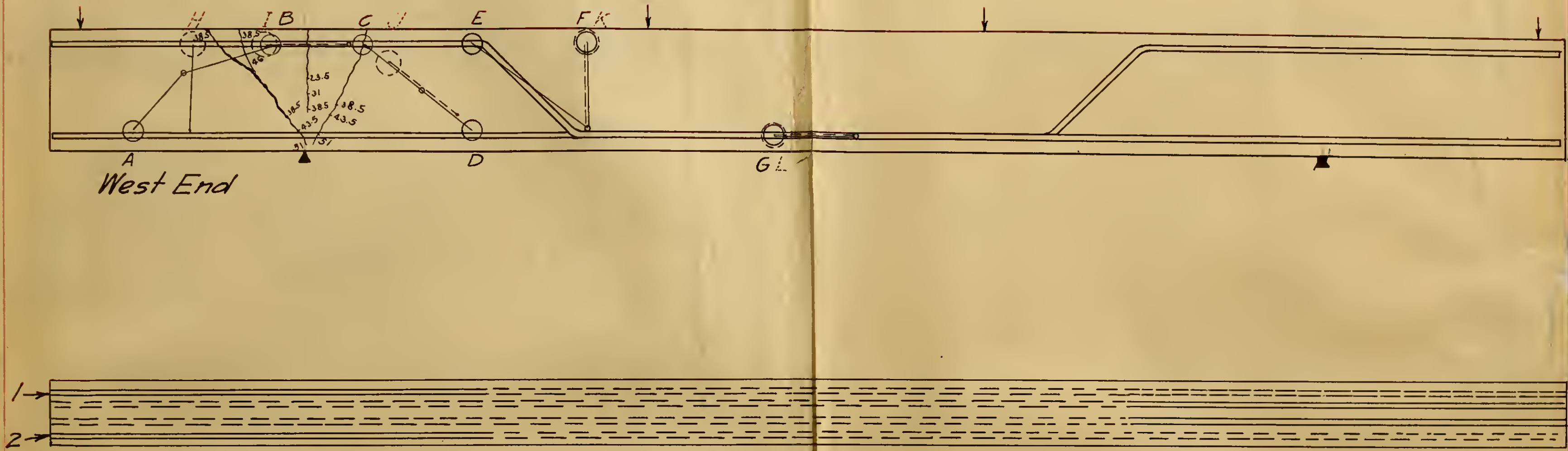
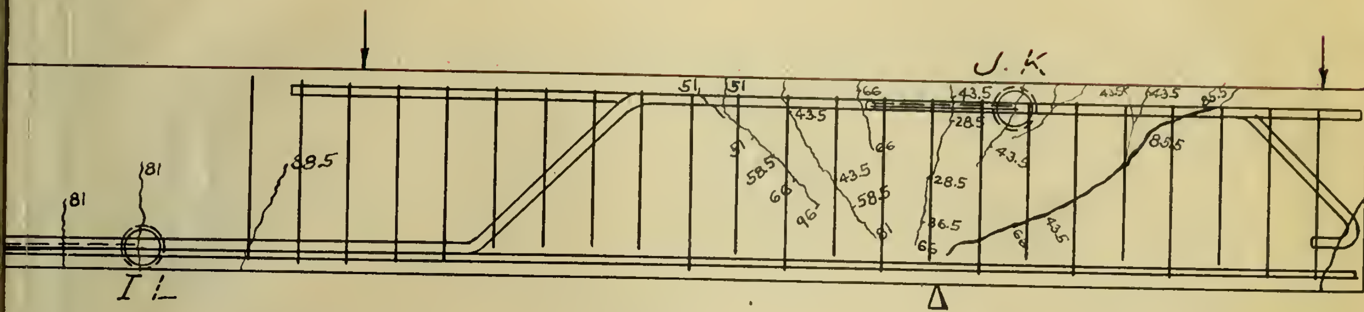
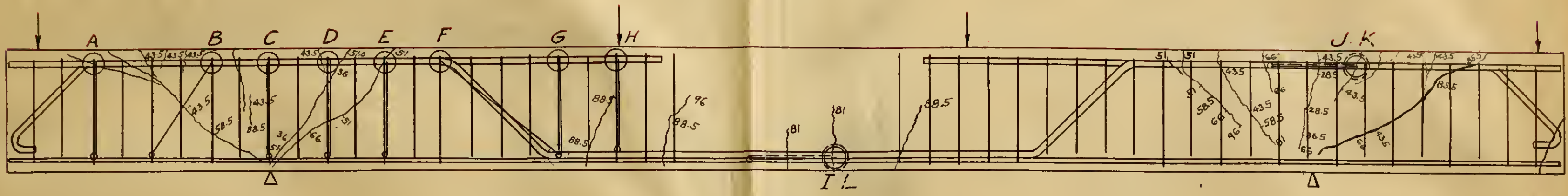

H
I B
C J
E
F K
A
D
G L
West End
38.5
38.5
46
23.5
31
38.5
38.5
43.5
43.5
51
51
1
2

Fig. 12


J. K
I L
81
81
38.5
51
51
66
43.5
28.5
43.5
43.5
85.5
85.5
58.5
66
96
43.5
58.5
81
28.5
36.5
65
43.5

West End

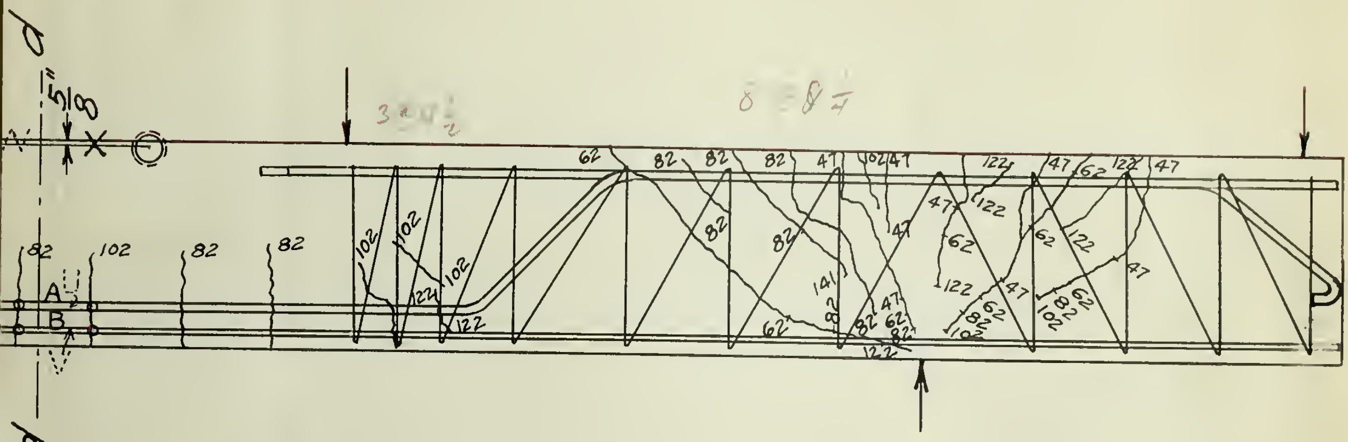
375.1

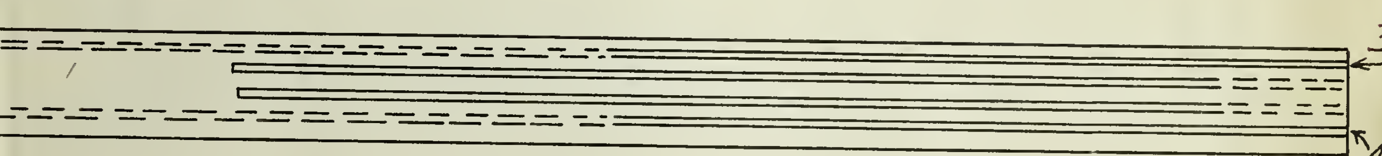


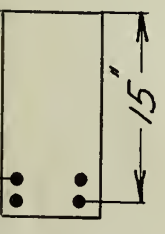
West End

Fig. 13

76.1







ction d-d

. 14

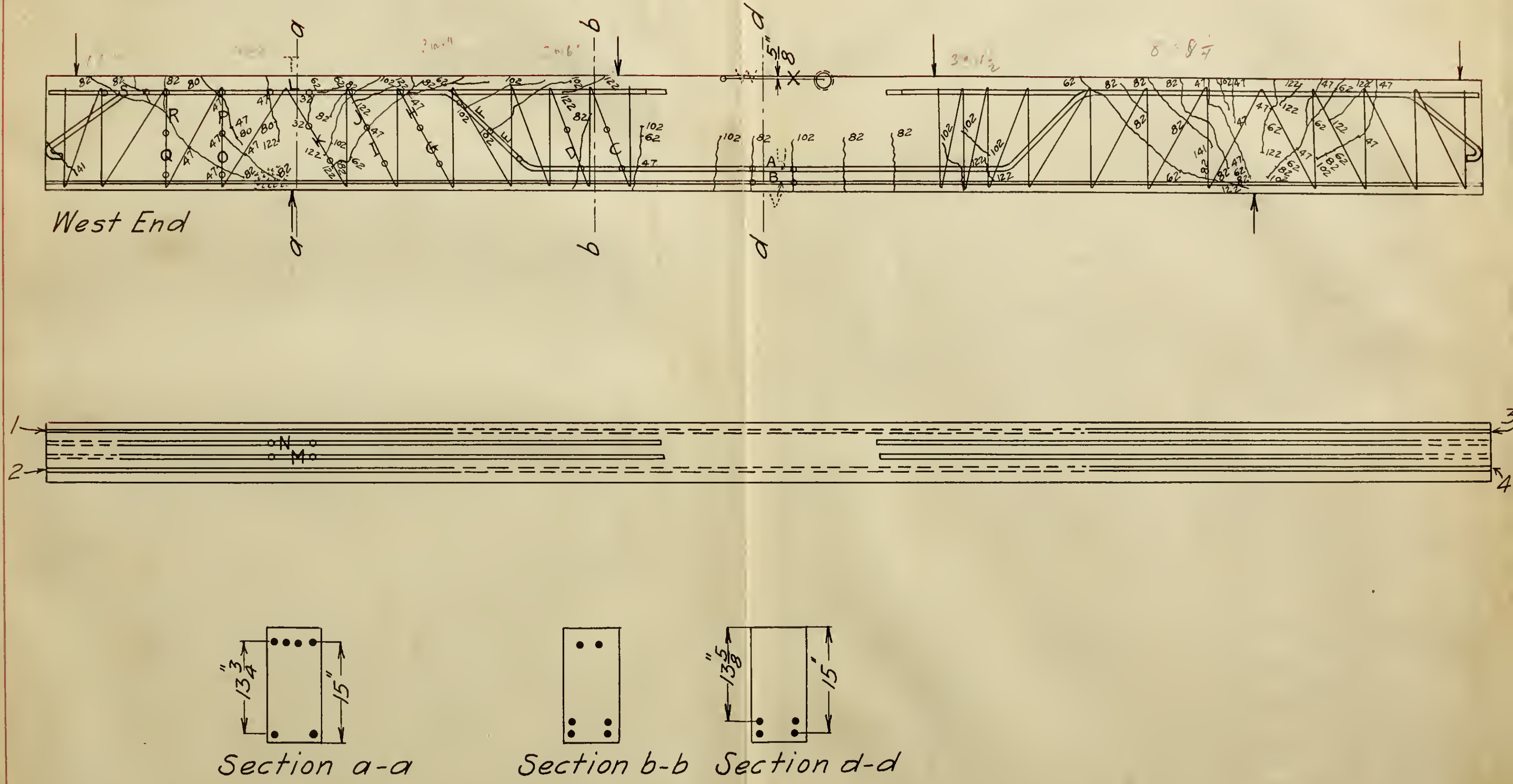
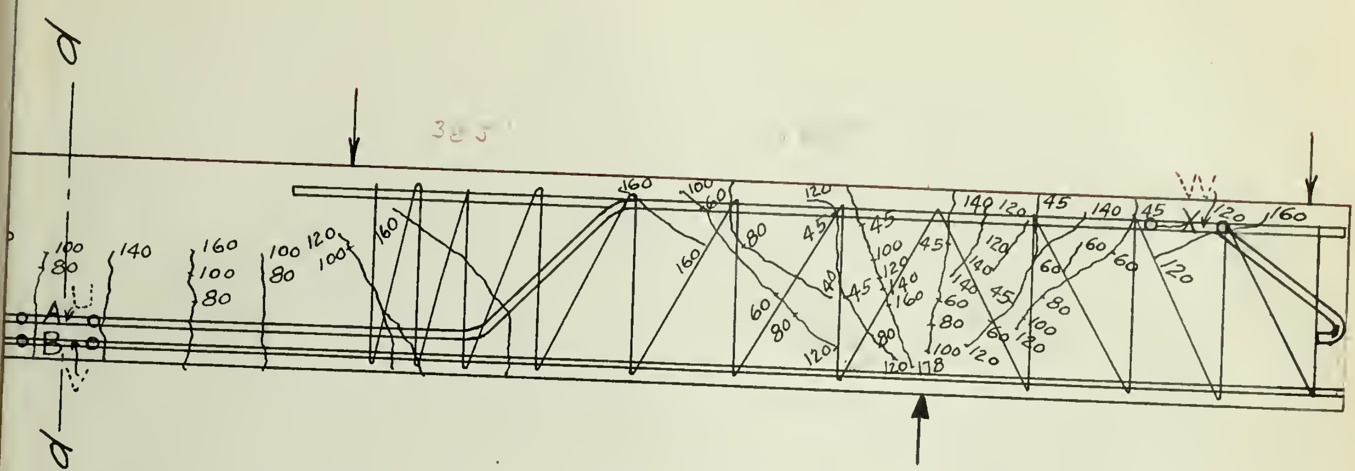
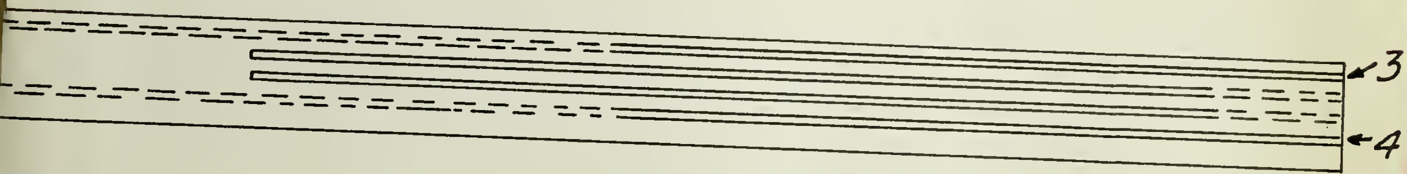

West End
Section a-a
Section b-b
Section d-d
$13\frac{3}{4}''$
$15''$
$13\frac{5}{8}''$
$15''$
$\frac{5}{8}''$

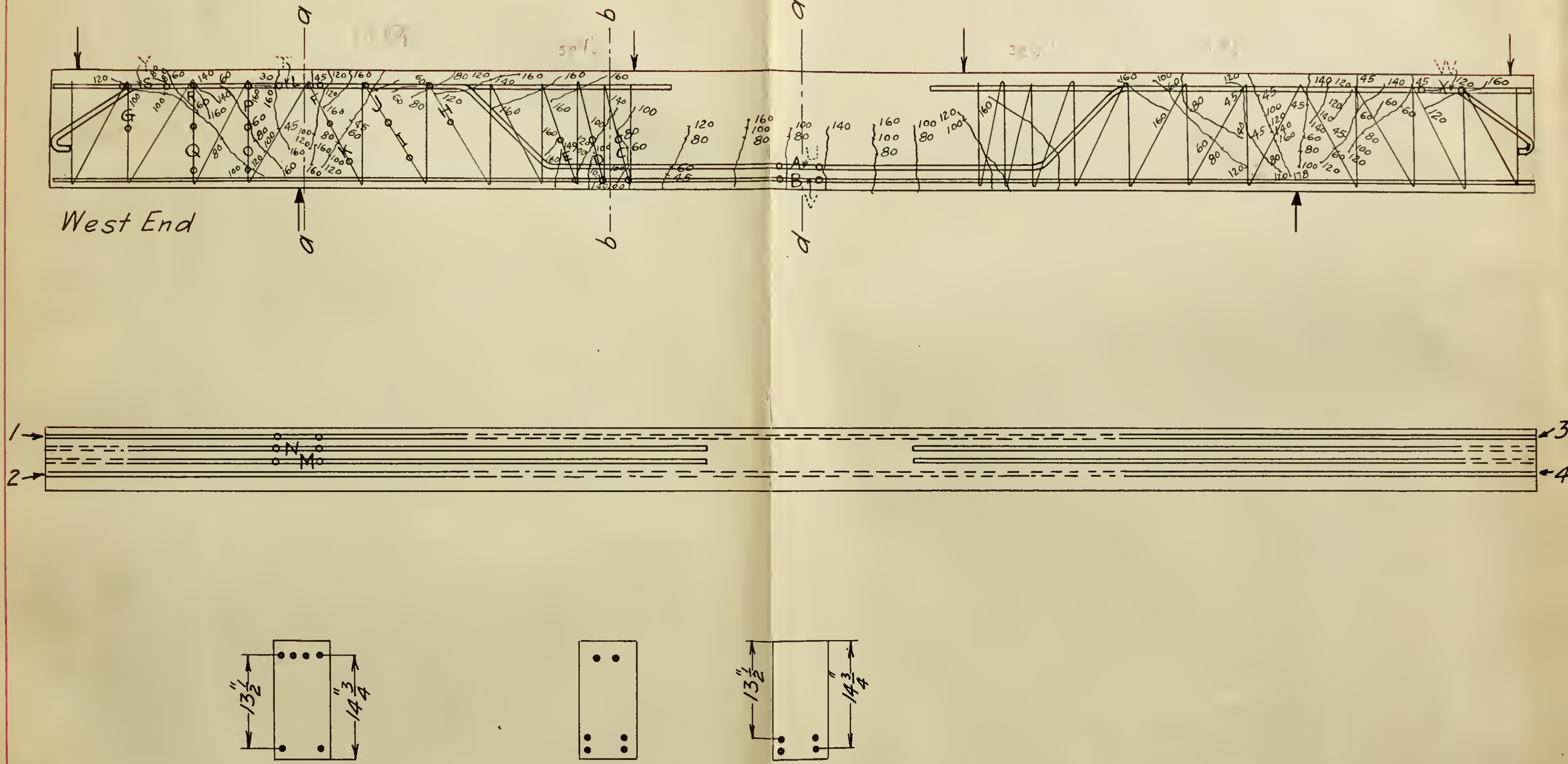
Fig. 14


d
d
A
B
U
W
X
100
80
140
160
100
80
100
80
120
100
160
160
100
60
160
80
60
80
120
45
45
100
120
140
160
45
80
120
178
140
120
45
120
140
60
80
100
120
45
140
45
60
60
60
80
100
120
120
160
120
120


3
4


$14\frac{3}{4}''$

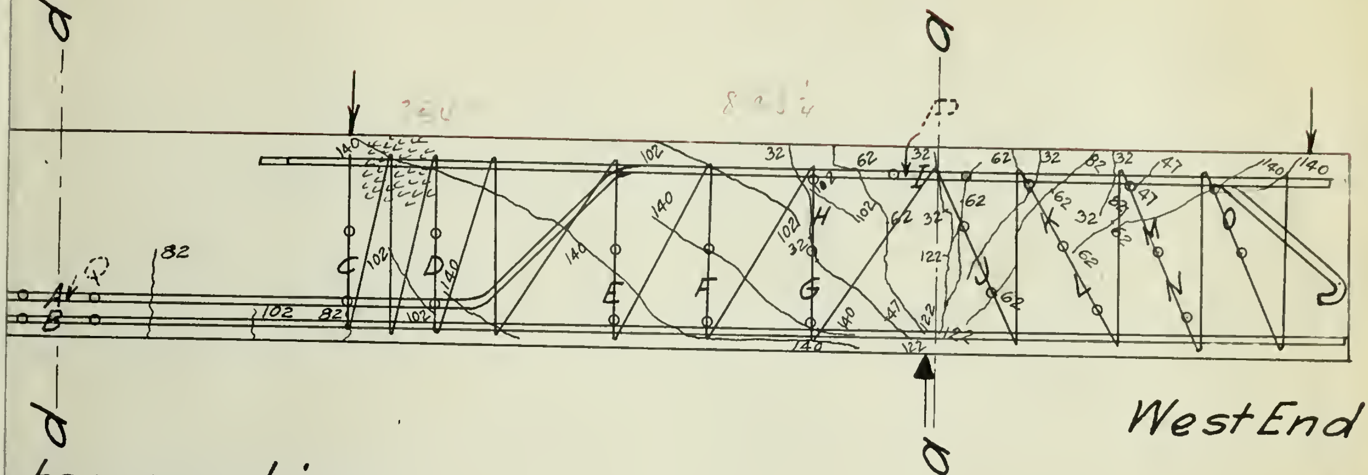
ction d-d



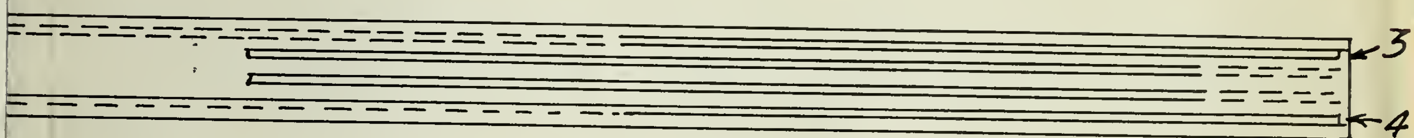
Section a-a Section b-b Section d-d

Fig. 15

76.5



ates crushing



ction d-d

. 16

376.5

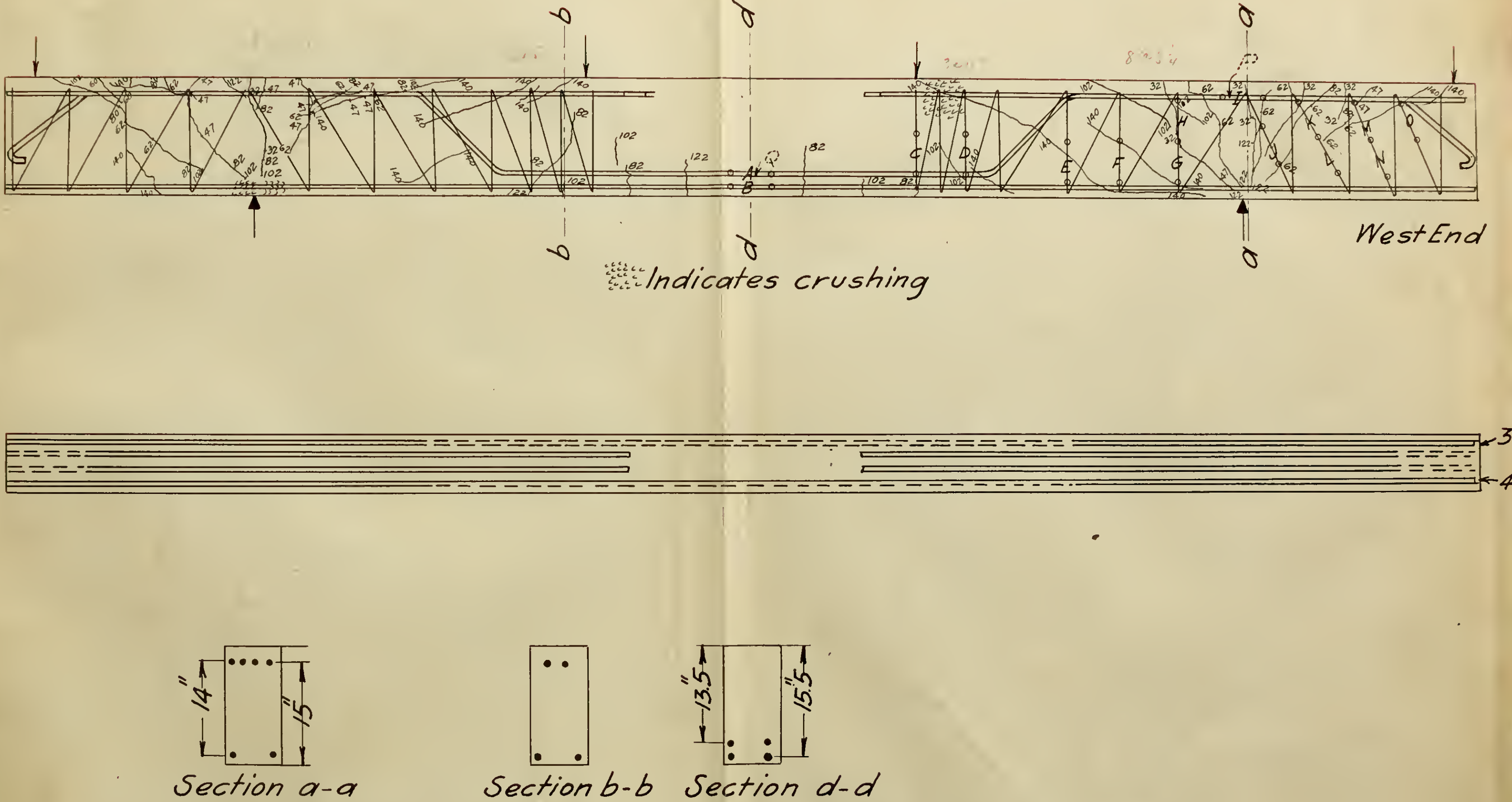
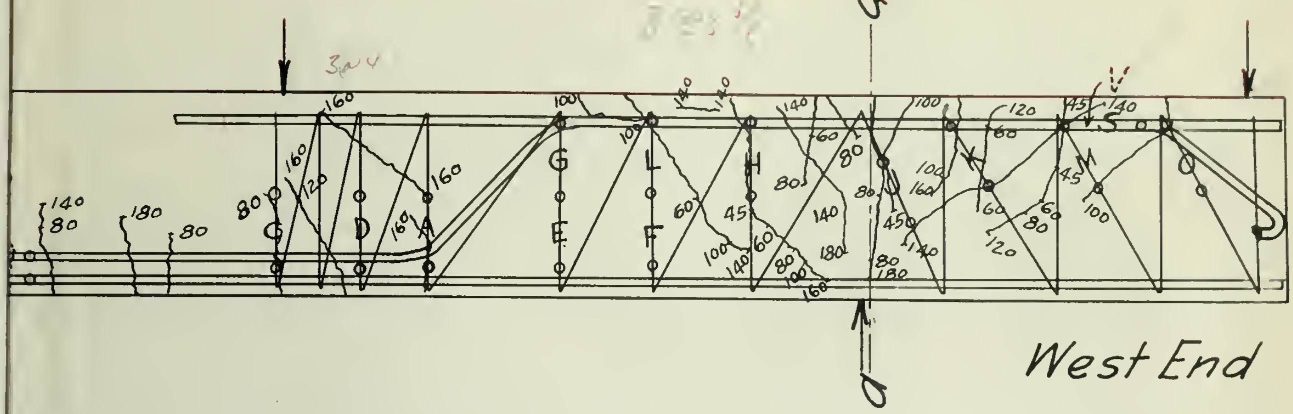
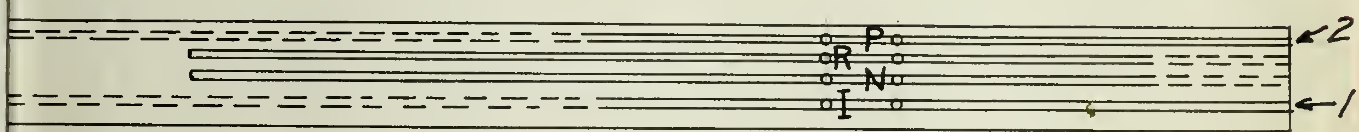
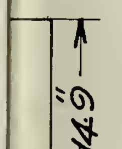

West End
Indicates crushing
3
4
14"
15"
13.5"
15.5"
Section a-a
Section b-b
Section d-d

Fig. 16

6.6.

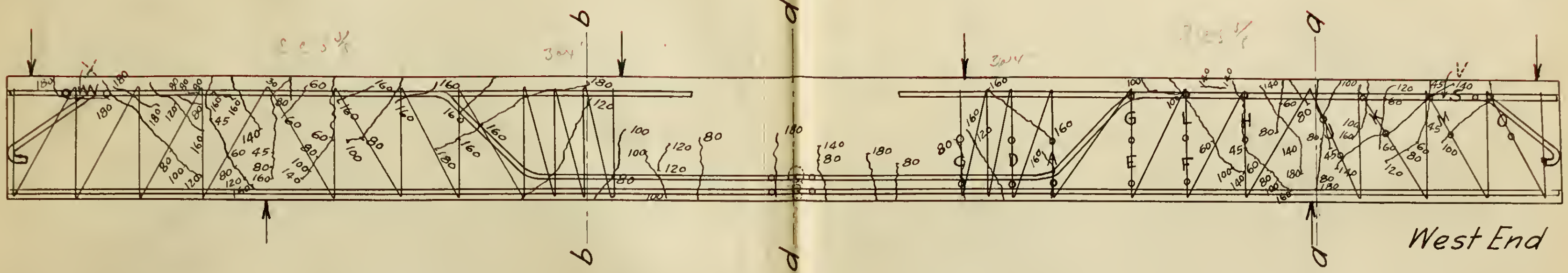


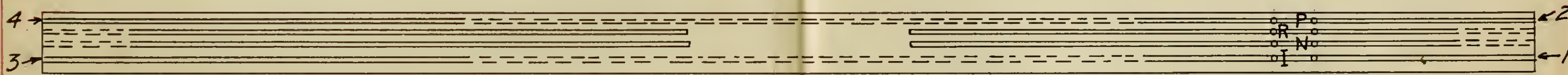


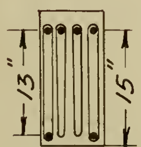


ion d-d

376.6

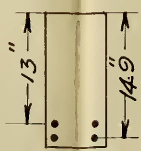






Section a-a

Section b-b



Section d-d

Fig. 17

OBSERVED DATA

OBSERVED DATA

371.2
(Tested as an over-hanging beam. See Fig. 7)

Load	$\frac{V}{b\ d}$	$\frac{V}{b\ jd}$	A	U	V	D	E	F	G	H	I
2300	4.8	5.4	.0997	.1412	.0715	.2118	.1890	.0668	.1722	.1083	.0663
15000	30	35	.0993 t 400	.1353 t 5900	.0718 c 300			.0659 t 900			.0656 t 700
30000	60	70	.0972 t 2500	.1263 t 14900	.0726 c 1100		.1889 t 100			.1093 c 1000	.0672 c 900
45000	91	104	.0994 t 300	.1102 t 31000	.0649 t 6 600	.2160 c 4 200	.1869 t 2100	.0620 t 4800	.1742 c 2000	.1102 c 1900	.0628 t 3 500
60000	121	139	.0972 t 2500	.1032 t 38000	.0679 t 3600	.2118 0000	.1872 t 1800	.0583 t 8500	.1749 t 2700	.1122 c 3900	.

Note: While taking the last series of readings, the load "dropped off" to 54 500 lb.

The blank spaces above indicate that no readings were taken .

c = compression

t = tension

All gage lengths 6 in.

OBSERVED DATA

371.2

(Tested as an over-hanging beam. See Fig.7)

Load	J	K	L	M	N	O	P	Q	R	S	T
2300	.2361	.1842	.2420	.1900	.1900	.2373	.2123	.1333	.1893	.1635	.1370
15000	.2362 c 100	.1829 t 1300	.2498 c 7800	.1896 t 400	.1879 t 2100	.2390 c 1700	.2113 t 1000	.1326 t 700	.1884 t 900	.1613 t 2200	.1333 t 3700
30000	.2352 t 900	.1836 t 600	.2412 t 800	.1945 c 4500	.1882 t 1800	.2382 c 900	.2133 c 1000	.1354 c 2100	.1885 t 800	.1486 t 14900	.1159 t 21100
45000	.2344 t 1700	.1829 t 1300	.2467 c 4700	.1845 t 5500	.1899 t 100	.2367 t 600	.2109 tr 1400	.1319 t 1400	.1871 t 2200	.1372 t 26300	.1012 t 35800
60000	.2325 t 3600	.1758 t 8400	.2438 c 1800	.1758 t 14200	.1867 t 3300	.2361 t 1200	.2122 t 100	.1349 c 1600	.1848 t 4500	.1034 ———	.0272 ———

Note: While taking the last series of readings, the load "dropped off" to 54 500 lb.

65400 Ultimate

c = compression

t = tension

All gage lengths 6 in.

OBSERVED DATA

371.2
(Tested as a simple beam . See Fig. 7)

Load	$\frac{V}{b\ d}$	$\frac{V}{b\ jd}$	V	A	B	C	D	E	F	G	H
2300	5	6	.0800	.1037	.1330	.1762	.2030	.1937	.0534	.1787	.1157
15000	58.5	70	.0673	.0957	.1339	.1810	.2070	.1925	.0530	.1793	.1150
30000	117	139	.0567	.0820	.1360	.1790	.2047	.1877	.0530	.1743	.1163
45000	175	209	.0437	.0724	.1370	.1821	.2030	.1900	.0530	.1767	.1145
49000 (44000)	191	228	Beyond range of instrument		.1370	.1783	.2000	.1910	.0523	.1770	.1120

Load	I	W	X	Y	Z	ϕ	Δ	θ	Σ
2300	0564	.0740	.1427	.1797	.1127	.0990	.1550	.1293	.1600
15000	.0577	.0717	.1423	.1800	.1087	.0942	.1535	.1273	.1590
30000	.0573	.0693	.1400	.1780	.1113	.0893	.1533	.1207	1590
45000	.0580	.0652	.1427	.1800	.1090	.0853	.1496	.1157	.1482
49000 (44000)	.0567	.0750	.1417	.1790	.1100	.0810	.1480	.1155	.1502

Note : During the last series of readings, the load "dropped off" to 44 000 lb.

Only the instrument readings are given above. All gage lengths 6 in.

OBSERVED DATA

372.1

Load	J	K	L	M	N	O	P	Q	R	S
2300	.1560	.2150	.1765	.2065	.2040	.1762	.1730	.1960	1697	.2467
15000	.1584	.2112	.1775	.2060	.2070	.1794	.1714	.1974	.1729	.2500
	c 2400	t 3800	c 1000	t 500	c 3000	t 3200	t 1600	c 1400	c 3200	c 3300
30000	.1549	.2066	.1750	.2080	.2089	.1744	.1717	.1920	.1717	.2490
	t 1100	t 8400	t 1500	c 1500	c 4900	t 1800	t 1300	t 4000	c 2000	c 2300
45000	.1465	.2000	.1710	.2105	.2007	.1707	.1687	.1897	.1667	.2437
	t 9500	t 15000	t 5500	c 4000	c 3700	t 5500	t 4300	t 6300	t 3000	t 3000
60000	.1369	.1975	.1732	.2097	.2046	.1703	.1613	.1803	.1563	.2383
	t 20000	t 17500	t 3300	c 3800	c 600	t 5900	t 11700	t 15700	t 13400	t 8400
80000	.1278	.1892	.1717	.2082	.2008	.1575	.1551	.1705	.1535	.2291
	t 28200	t 25800	t 4800	c 1700	c 800	t 18700	t 17900	t 25500	t 16200	t 17600
100000	.1236	.1832	.1677	2094	.1883	.1470	.1460	.1730	.1382	.2273
	t 32400	t 31800	t 8800	c 2900	t 15700	t 29200	t 27000	t 23000	t 31500	t 19400
110000 Ultimate										

c = compression

t = tension

All gage lengths 6 in.

OBSERVED DATA

372.1

Load	$\frac{V}{b\ d}$	$\frac{V}{b\ jd}$	A	B	C	D	E	F	G	H	I
2300	4.5	5.4	.1945	.0658	.2000	.22 50	.1822	.1889	.2120	.1875	.2350
15000	30	35.4	.1930	.0665	.1980	.2250	.1812	.1900	.2122	.1871	.2330
			t 1500	c 700	t 2000	0000	t 1000	c 100	c 200	t 400	t 2000
30000	59.5	70.8	.1927	.0640	.1978	.2250	.1792	.1890	.2120	.1846	.2268
			t 1800	t 1800	t 2200	0000	t 3000	c 100	0000	t 2900	t 8200
45000	89	116	.1865	.0640	.1977	.2250	.1790	.1865	.2118	.1757	.2225
			t 8000	t 1800	t 2300	0000	t 3200	t 2400	t 200	t 11800	t 12500
60000	119	142	.1797	.0647	.1937	.2255	.1775	.1890	.2114	.1703	.2177
			t 14800	t 1100	t 6300	c 500	t 4700	c 100	t 600	t 17200	t 17300
80000	158	189	.1717	.0632	.1862	.2077	.1772	.1894	.2100	.1616	.2117
			t 22800	t 2600	t 13800	t 17300	t 5000	c 500	t 2000	t25900	t23 300
100000	198	236	.1492	.0617	.1760	1897	.1742	.1882	.2072	.1573	.2030
			——	t 4100	t 24000	t 35300	t 8000	t 700	t 4800	t 30200	t 32000
110000	Ultimate										

c = compression

t = tension

All gage lengths 6 in.

OBSERVED DATA

372.2

Load	$\frac{V}{b\ d}$	$\frac{V}{b\ jd}$	A	B	C	D	E	F	G	H	I	J
2300	4.8	5.5	.2270	.2275	.1220	.0525	.1120	.1960	.0865	.1470	.1990	.2420
30000	60.5	72	.2220 t 5000	.2280 c 500	.1220 00000	.0507 t1800	.1120 0000	.1953 t 700	.0870 c 500	.1390 t 8000	.1820 t17000?	.2337 t 8300
45000	91	108	.2110 t16 000	.2280 0000	.1210 t1000	.0510 t1500	.1107 t1000	.1953 t1000	.0860 t 500	.1350 t12000	.1877 t11000	.2290 t13000
60000	121	144	.2037 t 23000	.2280 0000	.1195 t 2000	.0500 t 2500	.1110 t1000	.1930 t3000	.0857 t1500	.1300 t17000	.1820 t17000	.2243 t18000
80000	161	197	.1977 t29 000	.2267 t 1000	.1217 0000	.0507 t 1500	.1090 t 3000	.1863 t10000	.0857 t 500	.1230 t24000	.1747 t24000	.2140 t28000
100000	202	240	.1913 t 36000	.2260 t 2000	.1190 t 3000	.0507 t 1500	.1090 t3 000	.1810 t15000	.0843 t2500	.1157 t31000	.1690 t30000	.2133 t29000
103600	Ultimate											

c = compression

t = tension

All gage lengths 6 in.

OBSERVED DATA

372.2

Load	K	L	M	N	O	P	Q	R	S	T	U	V
2300	.2050	.1820	.2060	.1720	.2290	.1050	.1460	.1220	.1390	.0640	.1630	.2335
30000	.1977 t 7000	.1823 0000	.2075 c1500	.1750 c 2000	.2300 cl 000	.1040 t1000	.1483 c 2000	.1213 t 1000	.1335 t 5500	.0643 0000	.1630 0000	.2330 t 500
45000	.1953 t10000	.1830 c1000	.2070 c1000	.1747 c 3000	.2287 0000	.1040 t1000	.1477 c 2000	.1240 c 2000	.1227 t16000	.0610 t 3000	.1627 0000	.2310 t 2500
60000	.1897 t15000	.1820 0000	.2080 c2000	.1693 t3000	.2273 t 2000	.0980 t7000	.1440 t 2000	.1197 t 2000	.1170 t22000	.0517 t12000	.1630 0000	.2277 t 5500
80000	.1843 t21000	.1787 t3000	.2083 c2000	.1560 t16000	.2180 t11000	.0880 t17000	.1390 t 7000	.1120 t10000	.1090 t30000	.0480 t16000	.1605 t 2000	.2230 t10500
100000	.1763 t29000	.1590 t23000	.2080 c2000	beyond range Inst.	.1370 ———	.0510 ———	beyond range Inst.	.1090 t13000	.1020 t37000	—.0140 ———	.1583 t 5000	.2163 t17500
103600	Ultimate											

c = compression

t = tension

All gage lengths 6 in.

OBSERVED DATA

372.2

Load	W	X	Y	Z	Φ	Δ	θ	ψ
2300	.2275	.2040	.1730	.1630	.1033	.0850	.0720	.0760
30000	.2270 t 500	.1990 t 5000	.1660 t 7000	.1560 t 7000	.0990 t 4000	.0810 t 4000	.0700 t 2000	.0760 0000
45000	.2250 t 2500	.1953 t 9000	.1607 t12000	.1520 t11000	.0953 t 8000	.0780 t 7000	.0650 t 7000	.0720 t 4000
60000	.2187 t 8500	.1917 t12000	.1573 t16000	.1457 t17000	.0890 t14000	.0720 t13000	.0630 t 9000	.0680 t 8000
80000	.2117 t15000	.1873 t17000	.1520 t21000	.1390 t24000	.0840 t19000	.0680 t17000	.0570 t15000	.0600 t16000
100000	.2070 t20500	.1787 t25000	.1480 t25000	.1367 t26 000	.0780 t25000	.0620 t23000	.0550 t17000	.0550 t22000
103600	Ultimate							

c = compression

t = tension

All gage lengths 6 in.

OBSERVED DATA

373.1

LOAD	$\frac{V}{b\ d}$	$\frac{V}{bjd}$	A	B	C	D	E	F	G
2300	4.8	5.7	.0785	.0748	.1425	.2400	.1042	.0393	.0797
15000	31.2	37.2	.0787 c200	.0753 c500	.1457 c3200	.2327 t7300	.1027 t1500	.0370 t2300	.0777 t2000
30000	62.5	74.4	.0777 t800	.0747 t100	.1437 c1200	.2329 t7100	.1017 t2500	.0367 t2600	.0770 t2700
45000	93.6	112	.0745 t4000	.0733 t1500	.1437 c1200	.2330 t7000	.1007 t7000	.0370 t2300	.0770 t2700
60000	125	149	.0702 t8300	.0735 t1300	.1445 c2000	.2319 t8100	.1012 t3000	.0362 t3100	.0769 t2800
80000	167	198	.0672 t11300	.0725 t2300	.1432 c700	.2299 t10100	.1012 t3000	,0365 t2800	.0765 t3200
100000	208	248	.0607 t17800	.0697 t5100	.1454 c2900	.2277 t12300	.0992 t5000	.0382 t1100	.0757 t4000
114000	238	283	.0593 t19200	.0695 t5300	.1445 c2000	.2301 t9900	.1018 t2400	.0363 t3000	.0768 t2900

c = compression

t = tension

All gage lengths 6 in.

OBSERVED DATA

373.1

LOAD	H	I	J	K	L	M	N	O
2300	.2630	.2870	.1667	.0840	.0977	.2150	.2746	.2305
15000	.2634 c400	.2857 t1300	.1653 t1400	.0824 t1600	.0970 t700	.2144 t600	.2681 t6500	.2201 t10400
30000	.2678 c4800	.2840 t3000	.1617 t5000	.0824 t1600	.0964 t1300	.2102 t4800	.2615 t13100	.2195 t11000
45000	.2583 t4700	.2828 t4200	.1599 t6800	.0833 t700	.0940 t3700	.2033 t11700	.2586 t16000	.2139 t16600
60000	.2563 t6700	.2758 t11200	.1588 t7900	.0795 t4500	.0912 t6500	.1971 t17900	.2514 t23200	.2048 t25700
80000	.2508 t12200	.2694 t17600	.1521 t14600	.0805 t3500	.0905 t7200	.1877 t27300	.2424 t32200	.1994 t31100
100000	.2508 t12200	.2628 t24200	.1448 t21900	.0807 t3300	.0910 t6700	.1841 t30900	.2338 t40800	.1925 t38000
114000	.2434 t19600	.2594 t27600	.1421 t24600	.0808 t3200	.0898 t7900	.1801 t34900	.2331 t41500	.1918 t38700

OBSERVED DATA

373.1

LOAD	P	Q	R	S	T	U	V	W
2300	.2746	.2399	.1024	.2360	.0310	.1326	.2050	.0240
15000	.2727 t1900	.2399 0	.1024 0	.2397 c3700	.0269 t4100	.1320 t600	.2754 c400	.0224 t1600
30000	.2663 t8300	.2402 c300	.1014 t1000	.2352 t800	.0260 t5000	.1318 t800	.2689 t6100	.0214 t2600
45000	.2589 t15700	.2368 t3100	.1020 t400	.2386 c2600	.0260 t5000	.1336 c1000	.2699 t5100	.0220 t2000
60000	.2521 t22500	.2306 t9300	.0992 t3200	.2388 c2800	.0229 t8100	.1314 t1200	.2661 t8900	.0209 t3100
80000	.2435 t31100	.2274 t12500	.1012 t1200	.2313 t4700	.0229 t8100	.1298 t2800	.2581 t16900	.0209 t3100
100000	.2388 t34800	.2208 t19100	.0977 t4700	.2335 t2500	.0224 t8600	.1252 t7400	.2532 t22800	.0217 t2300
114000	.2366 t38000	.2158 t24100	.0955 t6900	.2341 t1900	.0168 t14200	.1294 t3200	.2518 23200t	.0208 t3200

373.2

Load	$\frac{V}{bd}$	$\frac{V}{bjd}$	A	B	C	D	E	F
2300	4.7	5.5	.1493	.1460	.1693	.0460	.1480	.0533
45000	90.7	108.0	.1483 t 1000	.1450 t 1000	.1700 c 700	.0460 00000	.1483 c 300	.0523 t1000
60000	121	144	.1430 t6300	.1457 t 300	.1683 t 1000	.0453 t 700	.1447 t3300	.0527 t 600
80000	161	197	.1417 t 700	.1460 0000	.1690 t 300	.0469 0000	.1387 t9300	.0553 c2000
100000	202	240	.1363 t 13000	.1438 t2200	.1690 t 300	.0440 t2000	.1377 t10300	.0513 t 2000
105000	212	279	.1330 t 16000	.1430 t3000	.1690 t 300	.0418 t4200	.1375 t10500	.0500 t3300

	G	H	I	J	K	L	M	N
2300	.0523	.1997	.1623	.1643	.1030	.0610	.1195	.0693
45000	.0555 c 3200	.1990 t 700	.1620 t 300	.1582 t6100	.1017 t 1300	.0603 t 700	.1077 t11800	.0563 t13000
60000	.0543 c 2000	.1973 t 2400	.1580 t 4300	.1547 t 9600	.1027 t 300	.0580 t 3000	.1050 t14500	.0493 t20000
80000	.0550 c 2700	.1977 t 2000	.1530 t9300	.1450 t19300	.1000 t 3000	.0540 t 7000	.0963 t23200	.0413 t28000
100000	.0512 t 1100	.1983 t1400	.1523 t10000	.1387 t25600	.0990 t 4000	.0460 t15000	.0920 t27500	.0355 t33800
105000	.0500	.1993	.1530	.1380	.0990	.0450	.0747	.0340

c = compression

All gage lengths 6 in.

t = tension.

OBSERVED DATA

373.2

Load	. O	P	Q	R	S	T	U
2300	.1107	.1803	.1060	.0972	.0450	.1687	.2247
45000	.0973 t13400	.1673 t13000	.1107 c 4700	.1000 c 2800	.0460 c 1000	.1667 t 2000	.2183 t 6400
60000	.0913 t19400	.1640 t16300	.1063 c 300	.1000 c 2800	.0457 c 700	.1637 t 5000	.2150 t 9700
80000	.0833 t27400	.1547 t25600	.1000 t 6000	.0983 c 1100	.0407 t 4300	.1503 t18400	.2070 t17700
100000	.0763 t34400	.1497 t30600	.0927 t13300	.0947 t 2500	.0367 t 8300	.1403 t28400	.2046 t20100
105000	.0770 t33700	t	.0480 ——	.0880 t 9200		.1500 t18700	.2013 t23400

	V	W	X	Y
2300	.1730	.0980	.0347	.1870
45000	.1717 t 1300	.0880 t10000	.0247 t10000	.1800 t 7000
60000	.1680 t 5000	.0867 t11300	.0180 t16700	.1707 t16300
80000	.1580 t15000	.0827 t15300	.1107 t24000	.1627 t24300
100000	.1530 t20000	.0813 t16700	.1060 t28400	.1530 t34000
105000	.0713 ——	.0723 t25700	out of range of instrument	

Note: After release of the load, the reading on"P" was .1755 .

c = compression

t = tension

All gage lengths 6 in.

OBSERVED DATA

376.1

Load	$\frac{V}{bd}$	$\frac{V}{bjd}$	A	B	C	D	E	F	G	H	I	J
2300	4.9	5.6	.8050	.8360	.2112	.1833	.0670	.1697	.1320	.5177	.1143	.2000
17300	37	42	.8077	.8374	.2122	.1822	.0692	.1672	.1302	.5174	.1139	.2009
			c 2700	c 1400	c 1000	t 1100	c 2200	t 2500	t 1800	t 300	t 400	c 900
32300	69	78	.8044	.8327	.2118	.1818	.0608	.1665	.1298	.5170	.1135	.1971
			t 600	t 3300	c 600	t 1500	t 6200	t 3200	t 2200	t 700	t 800	t 2900
47300	100	115	t.8012	.8292.	.2114	.1797	.0597	.1637	.1329	.5067	.1077	.1931
			t 3800	t 7800	c 200	t 3600	t 7300	t 6000	c 900	t 11000	t 6600	t 6900
62300	132	151	.7955	.8219	.2092	.1742	.0569	.1617	.1289	.4995	.1005	.1812
			t 9500	t 14100	t 2000	t 9100	t 10100	t 8000	t 3100	t 18200	t 13800	t 18800
82300	175	199	.7869	.8122	.2082	.1809	.0509	.1562	.1252	.4919	.0939	.1719
			t18100	t 23800	t 3000	t 2400	t 16400	t 13500	t 6800	t 25800	t 20400	t 28100
102300	218	248	.7782	.8049	.2072	.1659	.0445	.1549	.1192	.4866	.0819	.1602
			t26200	t 31100	t 4000	t 17400	t 22500	t 14800	t 12800	t 31100	t 32400	t 39800
122300	260	296	.7744	.7974	.2079	.1622	.0402	.1479	.1197	.5009	.0629	.1422
			t30600	t 38600	t 3300	t 21100	t 26800	t 21800	t 12300	t 16800	t 51400	t 57800
141100	Ultimate											

c = compression

t = tension

All gage lengths 6 in.

OBSERVED DATA

376.1

Load	K	L	M	O	P	Q	R	S	T	U	V
2300	.7870	.1847	.1957	.1282	.1440	.0507	.4250	.2080	.1320	.7827	.1000
17300	.7874	.1812	.1912	.1275	.1435	.0495	.4260	.2079	.1242	.7850	.0992
	c 400	t 3500	t 4500	t 700	t 500	t 1200	c 1000	t 100	t 7800	c 2300	t 800
32300	.7874	.1708	.1845	.1281	.1405	.0498	.4234	.1988	.1196	.7804	.0933
	c 400	t 13900	t 11200	t 100	t 3500	t 900	t 1600	t 9200	t 12400	t 2300	t 6700
47300	.7874	.1668	.1691	.1211	.1356	.0474	.4196	.1967	.1151	.7796	.0877
	c 400	t 17900	t 26600	t 7100	t 8400	t 3300	t 5400	t 11300	t 16900	t 3100	t 12300
62300	.7839	.1625	.1732	.1139	.1302	.0462	.4172	.1889	.1069	.7719	.0807
	t 3100	t 22200	t 22500	t 14300	t 13800	t 4500	t 7800	t 19100	t 25100	t 10800	t 19300
82300	.1749	.1549	.1659	.1048	.1199	.0429	.4485	.1842	.0979	.7639	.0772
	t 12100	t 29800	t29800	t 23400	t 24100	t 4100	t 14800	t 23800	t 34100	t18800	t 22800
102300	.7699	.1552	.1582	.0973	.1139	.0398	.4442	.1812	0902	.7569	.0694
	t 17100	t29500	t 37500	t 30900	t 30100	t 7400	t 19100	t 26800	t41800	t25800	t 30600
122300	.7600	.1502	.1499	.0899	.1079	.0372	.4409	.1775	.0835	.7500	.0620
	t 27000	t34 500	t 45800	t38 300	t 36100	t 9800	t 22400	t 30500	t 48500	t 32700	t 38000
141100	Ultimate										

c = compression

t = tension

All gage lengths 6 in.

OBSERVED DATA

376.1

Load	W			X		
	Inst. Reading	Total Def.	Unit Def.	Inst. Reading	Total Def.	Unit Def.
2300	.0000			.0000	~~.0002~~	
17300	.0006	.0012	.00008	.0001	.0002	.000013
32300	.0011	.0022	.00015	.0009	.0018	.00012
47300	.0020	.0040	.00026	.0018	.0036	.00024
62300	.0029	.0058	.00039	.0026	.0052	.00035
82300	.0039	.0078	.00052	.0039	.0078	.00052
102300	.0053	.0106	.00706	.0053	.0106	.00706
122300	.0065	.0130	.00866	.0075	.0150	.00100

Wire wound dials were used on gage lengths W and X . The length in each case was 15 inches. The deformations given above are expressed in inches.

OBSERVED DATA

376.2

Load	V / b d	V / b jd	A	B	C	D	E	F	G	H	I
2300											
2300	4.9	5.6	.2050	.1920	.1380	.1251	.1350	.2270	.1780	.1750	.0922
15000	31.8	37.0	.2048 t 200	.1921 c 100	.1410 c 3000	.1252 c 100	.1368 c 1800	.2275 c 500	.1793 c 1300	.1765 c 1500	.0985 c 6300
30000	64	74	.2003 t 4700	.1896 t 2400	.1433 c 5300	.1261 c 1000	.1351 c 100	.2279 c 900	.1799 c 1900	.1721 t 2900	.0973 c 5100
45000	95	101	.1981 t 6900	.1871 t 4900	.1426 c 4600	.1250 t 100	.1354 c 400	.2278 c 800	.1791 c 1100	.1737 t 1300	.0966 c 4400
60000	127	147	.1918 t 13200	.1834 t 8600	.1384 c 400	.1244 t 700	.1331 t 1900	.2221 t 4900	.1781 c 100	.1728 t 2200	.0968 c 4600
80000	169	197	.1891 t 15900	.1737 t 18300	.1332 t 4800	.1194 t 5700	.1297 t 5300	.1190 t 8000	.1771 t 900	.1692 t 5800	.0959 c 3700
100000	212	246	.1826 t 22400	.1683 t 23700	.1272 t 10800	.1141 t 11000	.1241 t 10900	.1129 t 14100	.1742 t 3800	.1709 t 4100	.0976 c 5400
120000	254	296	.1760 t 29000	.1587 t 33300	.1274 t 10600	.1077 t 17400	.1184 t 16600	.1084 t 18600	.1754 t 2600	.1687 t 6300	.0977 c 5500
140000	296	345	.1720 t 33000	.1534 t 38600	.1274 t 10600	.0979 t27200	.1116 t 23400	.0997 t 27300	.1730 t 5000	.1703 t 4700	.0954 c 3200
160000	339	394	.1645 t 40500	.1443 t 47700	.1240 t 14000	.0935 t 31600	.1102 t24 800	.0923 t 34700	.1697 t 8300	.1712 t 3800	.0880 t 4200
178000	Ultimate										

c = compression t = tension All gage lengths 6 in.

OBSERVED DATA

376.2

Load	J	K	L	M	N	O	P	Q	R	S
2300	.2275	.2230	.1920	.1422	.2260	.0594	.2272	.0893	.1360	.1765
15000	.2293 c 1800	.2248 c 1800	.1928 c 800	.1397 t 2500	.2235 t 2500	.0615 c 2100	.2282 c 1000	.0898 c 500	.1371 c 1100	.1776 c 1100
30000	.2289 c 1400	.2229 t 100	.1899 t 2100	.1361 t 6100	.2182 t 7800	.0603 c 900	.2271 t 100	.0923 c 3000	.1354 t 600	.1796 c 3100
45000	.2308 c 3300	.2224 t 600	.1848 t 7200	.1287 t 13500	.2118 t 14200	.0634 c 4000	.2260 t 1200	.0938 c 4500	.1356 t 400	.1773 c 800
60000	.2266 t 900	.2178 t 5200	.1811 t 10900	.1221 t 20100	.2066 t 19400	.0601 c 700	.2241 t 3100	.0924 c 3100	.1321 t 3900	.1693 t 6200
80000	.2227 t 4800	.2107 t 12300	.1741 t 17900	.1150 t 27200	.2001 t 25900	.0577 t 1700	.2184 t 8 800	.0934 c 4100	.1234 t 12600	.1631 t 13400
100000	.2196 t 7900	.1989 t 24100	.1696 t 22400	.1061 t 36100	.1896 t 36400	.0524 t 7 000	.2123 t 14900	.0856 t 3700	.1071 t 28900	.1582 t 18300
120000	.1181 t 9400	.1857 t 37300	.1647 t 27300	.0997 t 42500	.1836 t 42400	.0444 t 15000	.2047 t 22500	.0831 t 6 200	.0937 t 42300	.1570 t 19500
140000	.1157 t 11800	.1682 t ——	.1597 t 32300	.0916 t 50600	.1744 t 51600	.0274 t 32000	.1866 t 40600	.0784 t 10900	.0719 t ——	.1537 t 22800
160000	.1080 t 19500	.1333 t ——	.1553 t 36700	.0838 t 58400	.1660 t ——	.0060 t ——	.1532 ——	.0723 t 17000	beyond range Inst.	.1503 t 26200
178000	Ultimate									

c = compression

t = tension

All gage lengths 6 in.

OBSERVED DATA

376.2

Load	T	U	V	W	X	Y
2300	.1376	.1561	.1492	.0860	.1733	.1400
15000	.1365 t 1100	.1572 c 1100	.1472 t 2000	.0862 c 200	.1768 c 3500	.1402 c 200
30000	.1304 t 7200	.1534 t 2700	.1424 t 6800	.0851 t 900	.1766 c 3300	.1381 t 1900
45000	.1250 t 12600	.1457 t 10600	.1359 t 13300	.0794 t 6600	.1743 c 1000	.1347 t 5300
60000	.1184 t 19200	.1415 t 14600	.1318 t 17400	.0758 t 10200	.1671 t 6200	.1288 t 11200
80000	.1107 t 26900	.1345 t 22600	.1244 t24800	.0679 t 18100	.1581 t 15200	.1237 t 16300
100000	.1049 t 32700	.1283 t 27800	.1169 t 32300	.0599 t 26100	.1546 t 18700	.1176 t 22400
120000	.0969 t 40700	.1227 t33400	.1097 t 39500	.0550 t 31000	.1491 t 24200	.1127 t 27300
140000	.0919 t 45700	.1156 t 40500	.1019 t 47300	.0499 t36 100	.1447 t 28600	.1106 t 29400
160000	.0857 t 50100	.1118 t 44300	.0945 t54 700	.0435 t 42500	.1410 t 32300	.1108 t 29200
178000	Ultimate					

c = compression

All gage lengths 6 in.

t = tension

OBSERVED DATA

376.5

LOAD	$\frac{V}{b\,d}$	$\frac{V}{bjd}$	A	B	C	D	E	F	G
2300	4.8	5.5	.7382	.8582	.7033	.7517	.7439	.8397	.0097
17300	36.0	41.3	.7415 c3300	.8557 t2500	.6883 t15000	.7642 c6500	.7309 t13000	.8268 t12900	.0151 c5400
32300	67.4	77.1	.7372	-	.6892 t14100	.7583 c6600	.7337 t10200	.8310 t8700	.0147 c5000
47300	99.0	113	.7337 t4500	.8455 t12700	.6908 t12500	.7574 c5700	.7413 t2600	.8321 t7600	.0133 c3600
62300	130	149	.7303 t7900	-	.6928 t10500	.7639 c12200	.7376 t6300	.8359 t3800	.0133 c3600
82300	171	196	.7222 t16000	-	.6921 t11200	.7615 c9800	.7391 t4800	.8310 t8700	.0112 c1500
102300	213	245	.7214 t16800	.8257 t32500	.6906 t12700	.7613 c9600	.7390 t4900	.8280 t11700	.0099 c200
122300	255	292	.7229 t15300	.8315 t26700	.6888 t14500	.7623 c10600	.7415 t2400	.8236 t16100	.0103 c600
139800	291	334	.7228 t15400	.8258 t32400	.6968 t6500	.7626 c10900	.7279 t6000	.8232 t16500	-

c = compression

t = tension

All gage lengths 6 in.

OBSERVED DATA

376.5

LOAD	H	I	J	K	L	M	N	O
2300	.6973	.7329	.5914	.5322	.6287	.5549	.5575	.6246
17300	.7007	.7241	.5916	.5305	.6264	.5535	.5580	.6227
	c3400	t8800	c200	t1700	t2300	t1400	c500	t1900
32300	.6976	.7267	.5992	.5350	.6330	.5597	.5645	.6315
	c300	t6200	c7800	c2800	c4300	c4800	c7000	c6900
47300	.6965	.7197	.5889	.5278	.6257	.5539	.5589	.6216
	t800	t13200	t2500	t4400	t3000	t1000	c1400	t3000
62300	.6937	.7137	.5886	.5254	.6236	.5517	.5581	.6217
	t3600	t19200	t2800	t6800	t5100	t3200	c600	t2900
82300	.6949	.7075	.5860	.5184	.6182	.5489	.5549	.6206
	t2400	t25400	t5400	t13800	t10500	t6000	t2600	t4000
102300	.6904	.7020	.5854	.5147	.6154	.5147	.6154	.5503
	t6900	t30900	t6000	t17500	t13300	t4600	t2400	c200
122300	.6925	.6962	.5807	.5067	.6096	.5474	.5544	.6194
	t4800	t36700	t10700	t25500	t19100	t7500	t3100	t5200
139800	.6904	.6858	.5802	-	.6025	.5475	.5549	.6113
	t6900	t47100	t11200		t26200	t7400	t2600	t13300

OBSERVED DATA

376.5

Load	P	Q
2300	.5994	.4988
17300	.5995	.4989
	c100	c100
32300	.5997	.5031
	c300	c4300
47300	.5879	.5014
	t11500	c2600
62300	.5804	.4949
	t19000	t3900
82300	.5776	.4876
	t21800	t11200
102300	.5726	.4891
	t26800	t9700
122300	.5699	.4827
	t29500	t16100
139800	-	.4837
		t15100

OBSERVED DATA

376.6

LOAD	V / b d	V / bjd	A	B	C	D	E	F
2300	4.8	5.5	.0777	.2660	.1610	.1960	.0850	.2240
15000	31.2	36.8	.0762 t1500	.2645 t1500	.1595 t1500	.1932 t2800	.0842 t800	.2165 t7500
30000	62.4	71.6	.0762 t1500	.2563 t9700	.1602 t800	.1930 t3000	.0882 c3200	.2171 t6900
45000	93.6	107	.0761 t1600	.2565 t9500	.1584 t2600	.1934 t2600	.0881 c3100	.2175 t6500
60000	125	143	.0764 t1300	.2459 t20100	.1617 c700	.1920 t4000	.0844 t600	.2202 t3800
80000	167	191	.0773 t400	.2504 t15600	.1586 t2400	.1926 t3400	.0863 c1300	.2239 t 100
100000	209	239	.0791 c1400	.2407 t25300	.1593 t1700	.1970 c1000	.0883 c3300	.2197 t4300
120000	249	287	.0784 c700	.2363 t29700	.1581 t2900	.1924 t3600	.0891 c4100	.2238 t200
140000	291	334	.0768 t900	.2310 t35000	.1605 t500	.1918 t4200	.0885 c3500	.2223 t1700
160000	333	382	.0753 t4400	.2274 t38600	.1600 t100	.1910 t5000	.0883 c3300	.2223 t1700
180000	375	430	.0690 t8700	.2206 t45400	.1600 t100	.1890 t7000	.0870 c2000	.2236 t400

c = Compression

t = Tension

All gage lengths 6 in.

OBSERVED DATA

376.6

LOAD	G	H	I	J	K	L	M	N
2300	.2000	.2436	.1140	.1800	.2300	.1577	.1460	.1487
15000	.1935 t6500	.2387 t4900	.1067 t7300	.1782 t1800	.2295 t500	.1572 t500	.1452 t800	.1445 t4200
30000	.1995 t500	.2381 t4500	.1038 t10200	.1785 t1500	.2231 t6900	.1585 c800	.1442 t1800	.1385 t10200
45000	.1991 t900	.2385 t5100	.0985 t15500	.1775 t2700	.2257 t4300	.1564 t300	.1420 t4000	.1345 t14200
60000	.1962 t3800	.2359 t7700	.0886 t25400	.1757 t4300	.2212 t8800	.1564 t1300	.1387 t7300	.1272 t21500
80000	.1963 t3700	.2382 t5400	.0829 t31100	.1745 t5500	.2196 t10400	.1563 t1400	.1340 t12000	.1450 t26800
100000	.1990 t1000	.2334 t10200	.0751 t38900	.1708 t9200	.2121 t17900	.1545 t3200	.1273 t18700	.1131 t35600
120000	.1978 t2200	.2348 t8800	.0678 t46200	.1671 t12900	.2051 t24900	.1551 t2600	.1224 t23600	.1048 t43900
140000	.1975 t2500	.2297 t13900	.0620 t52000	.1435 t36500	.1978 t32200	.1495 t8200	.1158 t30200	.0987 t50000
160000	.1977 t2300	.2274 t16200	.0554 t58600	.1585 t21500	.1894 t40600	.1510 t6700	.1100 t36000	.0907 t58000
180000	.1980 t2000	.2286 t15000	—	.1460 t34000	.1746 t55400	.1510 t6700	.1060 t40000	—

OBSERVED DATA

376.6

LOAD	O	P	Q	R	S	V	W	X
2300	.1560	.2640	.2270	.2200	.2700	.2073	.2020	.1570
15000	.1528 t3200	.2635 t500	.2243 t2700	.2165 t3500	.2685 t1500	.2055 t1800	.2008 t1200	.1552 t1800
30000	.1575 c1500	.2611 t2900	.2217 t5300	.2088 t11200	.2664 t3600	.2035 t3800	.2015 t500	.1561 t900
45000	.1508 t5200	.2568 t7200	.2190 t8000	.2165 t3500	.2635 t6500	.1998 t7500	.1961 t5900	.1520 t5000
60000	.1490 t7000	.2512 t12800	.2162 t10800	.2005 t19500	.2592 t10800	.1967 t10600	.1897 t12300	.1516 t5400
80000	.1450 t11000	.2459 t18100	.2179 t9100	.1959 t24100	.2576 t12400	.1918 t15500	.0870 t15000	.2499 t7100
100000	.1523 t3700	.2361 t27900	.2134 t14600	.1862 t33800	.2458 t24200	.1903 t17000	.0830 t19000	.2434 t13600
120000	.1471 t8900	.2308 t33200	.2070 t20000	.1808 t39200	.2505 t19500	.1811 t26200	.0791 t22900	.2388 t18200
140000	.1455 t10500	.2197 t44300	.2030 t24000	.1717 t48300	.2415 t28400	.1765 t30800	.0728 t29200	.2355 t21500
160000	.1430 t13000	.2194 t44600	.1997 t27300	.1647 t55300	.2426 t27400	.1713 t36000	.0650 t37000	.2336 t23400
180000	.1440 t12000	.2066 t57400	.1976 t29400	.1216 t98400	.2376 t32400	.1690 t38300	.0670 t35000	.2326 t24400